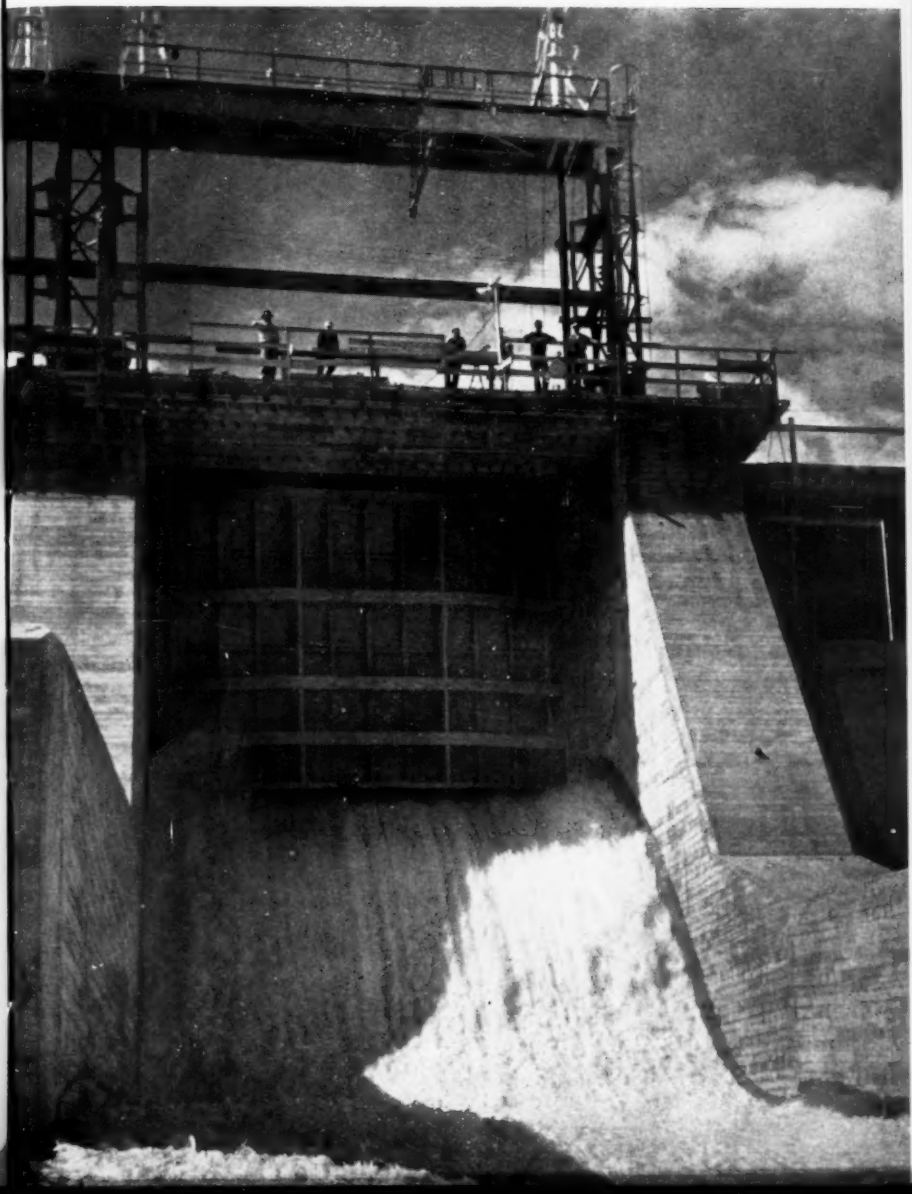
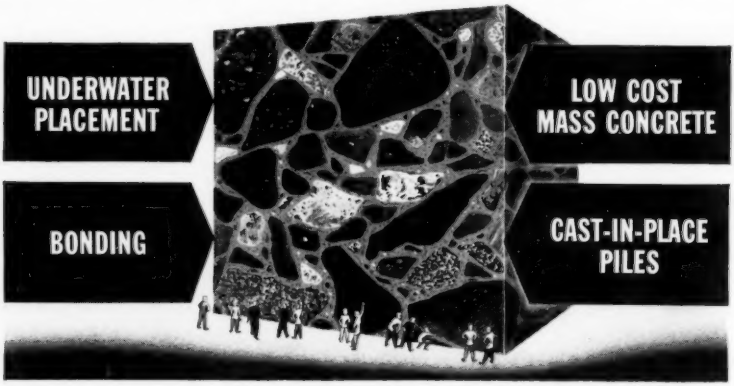
ACI News Letter

Vol. 22 No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1950



PREPAKT CONTRIBUTIONS TO CONCRETE CONSTRUCTION



UNDERWATER PLACEMENT—Prepakt methods and materials make practical the underwater placement of high-strength concrete—without need for unwatering operations or use of additional cement.

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ACI

News Letter Contents

November 1950

On the Cover—One of the two sluiceways at the Hydro-Electric Power Commission of Ontario's Stewartville development on the Madawaska River, Canada. These spillways, with their 35-ft steel gates, will discharge excess river flows into a concrete-lined, high-water channel. Baffle piers have been constructed to assist in de-energizing the water before it re-enters the river after a 60-ft fall.

Courtesy Hydro-Electric Power Commission of Ontario

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Washington Meeting Attendance Surpasses All Previous Regional Meetings

ACI's Washington Regional Meeting, Washington, D. C., Oct. 24-26, surpassed all previous ACI regional meeting attendance figures when 322 registrants crowded into three interest-getting technical sessions, with an average attendance at each session of 225-250. Registration matched or exceeded the registration at 14 of ACI's national conventions.

With most of the JOURNAL on the presses, while the meeting was still in progress, the December *News Letter* will feature the highlights of the complete meeting which was under the chairmanship of E. W. Bauman, Managing Director, National Slag Assn.

The three-day event included three technical sessions with an inspection trip to the Chesapeake Bay Bridge now under construction concluding the conference. The technical program included "Design of Concrete Mixtures for Interior of Dams," Byram W. Steele, Office of Chief of Engineers, Dept. of the Army; "Precautions in Finishing Pavements Made of Air-Entraining Concrete," Charles W. Allen, Research Engineer, Ohio State Highway Dept.; "Effective Sealing of Joints in Concrete Pavements," Harold F. Clemmer, Engineer of Materials, District of Columbia; "Investigation of Dowels and Other Load Carrying Devices for Transverse Joints of Concrete Pavements," Henri Marcus, Bureau of Yards and Docks, Dept. of the Navy.

An open forum on ready-mixed concrete, with a panel of experts in the ready-mix industry discussing questions from the floor, was featured at the Wednesday morning session. A film on the construction of the prestressed Walnut Lane Bridge in Philadelphia was also presented.

Topics covered in the third technical session included "A Traffic Test to Determine the Effects of Heavy Loads on Concrete Pavements," Fred Burggraf, Associate Director, Highway Research Board; "Application of Machine Technique to Concrete Housing Construction," William V. Reed, Vice-President, Ibec Housing Corp.; "Construction of the Chesapeake Bay Bridge," Robert A. Gilmore, Project Engineer, J. E. Greiner Co.; and "Properties of 'No-Fines' Air-Entraining Concrete," Rudolph Valore, Jr., National Bureau of Standards.

See the December JOURNAL for a complete review of the Washington meeting.

Courtesy San Francisco Convention and Tourist Bureau

So you're going to San Francisco?

Golden Gate Bridge

San Francisco is linked to the East Bay area in one direction and the Marin Peninsula in the other by two of the world's largest bridges, the San Francisco-Oakland Bay Bridge and the Golden Gate Bridge. The San Francisco-Oakland Bay bridge was described in the October JOURNAL. The combined cost of these two structures totaled more than $100,000,000.

The Golden Gate Bridge, containing the longest single span in the world, is the second largest bridge in the world and is a link in U. S. Highway No. 101.

From bridge-head to bridge-head the bridge is 6400 ft, but from portal to portal it is 8940 ft. The total distance shore to shore, *i.e.*, the width across the Golden Gate along the center-line of the bridge is 5537 ft. The distance from the Marin shore to north pier is 52 ft, and from the San Francisco shore to south pier is 1105 ft. The main span distance, center to center of piers, is 4200 ft, longest in the world. The clearance height is 210 ft at the piers, increasing to 220 ft at the center, measured from the mean higher high water and under maximum deflection.

The most prominent features of the Golden Gate Bridge are the two great steel towers carrying the cables which support the structure. These towers are 746 ft above mean high water, or 191 ft taller than the Washington Monument. The towers are 846 ft high, measured from the base of the San Francisco pier, the highest and largest bridge towers in the world.

There are two cables for the bridge, resting on cast steel saddles at the tops of the towers and anchored on shore. Each cable is 7660 ft long and 36 1/2 in. in diameter, containing 27,572 steel wires, laid parallel to each other and strung from anchor to anchor in one continuous operation, compacted by a special device and wrapped with galvanized steel wire. Each cable weighs 11,000 tons and the two cables required 80,000 mi. of wire. The bridge required enough structural steel and cable steel to load a freight train 20 mi. long.

The Golden Gate Bridge is another great monument to engineering vision and "know-how." Where else but San Francisco could you compare the world's two largest bridges?

Who's Who This Month

Committee 805

ACI Committee 805 under the chairmanship of W. L. Chadwick, Southern California Edison Co., Los Angeles, submits "Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure," p. 185. The committee includes J. A. McCrory, Shawinigan Engineering Co., Ltd., Montreal, Que., Canada, and R. B. Young, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada.

The committee, organized in 1942, presents the report for discussion with a view to its consideration for adoption as an ACI recommended practice at the 47th annual convention in San Francisco, Feb. 20-22, 1951.

The proposed standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete.

Fred Uziel

Making his first appearance in JOURNAL pages, Fred Uziel, staff engineer, Preload Corp., New York, N. Y., describes a direct, rapid and accurate design method for "Determining Optimum Cross Sections for Prestressed Concrete Girders," p. 197.

He has been connected with the design of prestressed tanks and bridges for the corporation. The Preload Corp., the construction arm of Preload Enterprises, Inc., is currently completing construction of the first prestressed concrete bridge in this country, the Walnut Lane Bridge in Philadelphia.

Mr. Uziel studied at the National Metsovion Polytechnion of Athens, Greece, and graduated from Rensselaer Polytechnic Institute with a BS in civil engineering. He received an MS in building engineering and construction from the Massachusetts Institute of Technology.

C. O. Crane

"Steam Curing Protects Winter Concreting," p. 213, is the second paper by C. O. Crane, engineer, U. S. Bureau of Reclamation, Enders Dam, Enders, Neb., about construction activities at Enders Dam (See June 1949 JOURNAL, p. 733).

Mr. Crane is acting construction engineer for the Enders project and has been field engineer on the project since 1945. From 1942-45 he was head of the Public Works Planning Section, Bureau of Yards and Docks, Dept. of the Navy, Washington, D. C.

He joined the U. S. Bureau of Reclamation in 1934, first as a concrete inspector, and later made tempreature studies of concrete, investigated the use of vacuum processed concrete and developed an absorptive concrete form lining. He graduated from Utah State College and holds MS and CE degrees from the University of Colorado.

Becla Velutini

Becla Velutini, Caracas, Venezuela, a Junior Member of the Institute, presents "Analysis of Continuous Circular Curved Beams," p. 217. Miss Velutini, now in private structural design practice, graduated from the Universidad Central de Venezuela in 1945, and also studied at the University of Michigan. Prior to entering private practice, she worked on structural designs for the Ministerio de Obras Publicas, Division de Calculo, Direccion de Edificios, in Venezuela.

J. Neils Thompson and Phil M. Ferguson

J. Neils Thompson and Phil M. Ferguson, Professor of Civil Engineering and Chairman, Civil Engineering Dept., respectively, University of Texas, Austin, are co-authors of "Shear Resistance of Tile-Concrete Floor Joists," p. 229.

Professor Thompson is in charge of the materials laboratories of the civil engineering department and is also Director of the Off-Campus Research Center. As Director he is in charge of the coordination and development of the Center which was established at the International Minerals and Chemical Co. magnesium plant in Austin. The plant was taken over by the University of Texas and about 17 research laboratories established for research in geology, physics, electronics and engineering.

After graduation from the University of Texas in 1935, Professor Thompson worked for the Materials and Tests Division, Texas Highway Dept. He joined the university staff in 1941.

Author of a number of papers in technical publications, Professor Thompson was Director of the Texas Society of Professional Engineers, 1944-46. He is also a member of ACI, ASCE, Asphalt Paving Technologists, Society for Experimental Stress Analysis, Highway Research Board and the American Society for Engineering Education.

Professor Ferguson presented a paper on three-dimensional beam-and-girder framing in the September issue (p. 61) and returns this month as a co-author.

An ACI Member since 1930, he is a member of Committee 115, Research, with his chief fields of interest in frame analysis and reinforced concrete. Over the past ten years research on stress distribution in reinforcing steel, especially as modified by cracks, and questions of bond strength and bond stress distribution have occupied more and more of his time.

After graduation from the University of Texas in 1922 and master's degrees from the University of Texas in 1923 and the University of Wisconsin in 1924, he spent several years with Dwight P. Robinson and Co. of New York City in structural design of power plants and industrial buildings and construction of hotels and apartment buildings.

Professor Ferguson returned to the University of Texas as associate professor in the field of structures and materials in 1928. He was raised to the rank of professor in 1939 and was made chairman of the civil engineering department in 1943. He has also been consultant on various projects.

Alfred Parme

The mathematical discussion, "Solution of Difficult Structural Problems by Finite Differences," p. 237, is by Alfred Parme, engineer, Structural and Railways Bureau, Portland Cement Assn., Chicago, Ill.

A member of ACI Committee 314, Rigid Frame Bridges, Mr. Parme is also a member of the ASCE Committee on thin shell design and the author of several papers on structural design.

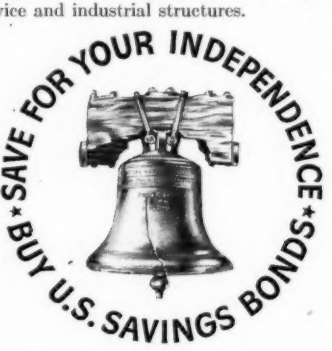
After graduation from Cornell University in 1935 in civil engineering he was employed by the Phoenix Engineering Co. on the design of hydraulic structures. He joined the Corps of Engineers in 1937 as assistant engineer with duties involving the structural design of flood control appurtenances such as conduits, intakes and dams. In 1940 Mr. Parme joined the PCA staff, collaborating in the preparation of structural data and reports on the design of reinforced concrete structures.

Lawrence R. Hjorth

Lawrence R. Hjorth, Edwards and Hjorth, Consulting Engineers, New York, died recently in Calais, Me.

After graduation from the University of Washington in civil engineering, he was employed by the Milwaukee and St. Paul Railroad, Turner Construction Co. and joined the H. G. Balcom consulting firm in 1919. In 1933 he became a partner in the firm of H. G. Balcom and Associates, remaining there until the firm of Edwards and Hjorth was formed.

During his association with H. G. Balcom he had charge of design of various projects, including the Chrysler Building; Waldorf Astoria; Cathedral of Learning, Pittsburgh University; and Radio City Music Hall. Under the firm name of Edwards and Hjorth he was associated in the design of the Crowell-Collier Building, U. N. Secretariat Building and Esso Building in New York; the U. S. Steel-Mellon Bank and Alcoa Building in Pittsburgh; Chile Steel Mill in Concepcion, Chile; and many other educational, religious, service and industrial structures.



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Positions and Projects — ACI Members

Research

The assignment of Committee 115, Research, "is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives." The committee has attempted to fulfill this assignment through the research sessions at the annual conventions, through the distribution of the annual compilation of research projects underway at the various laboratories and through limited correspondence among committee members.

From the continuing interest of the ACI membership in the activities of Committee 115, it is evident that real needs are being met. It would be presumptuous, however, to assume that the committee has reached the zenith of its possible usefulness; there must be various respects in which increased service can be rendered. Reactions and suggestions for greater usefulness are earnestly requested; perhaps there are laboratories or projects which have been overlooked in sending out recent requests for information; perhaps there are questions that should be raised or additional activities that should be started. Since the committee is anxious to have complete coverage of all research projects in its next compilation, all persons having suggestions or knowing of omissions are urged to initiate correspondence at once. No publicity will be given any information without the permission of the correspondent. Address your suggestions and inquiries to:

Prof. George W. Washa
Secretary ACI Committee 115
Engineering Building
University of Wisconsin
Madison 6, Wis.

Bulletin Technique de la Suisse Romande

This French-language Swiss technical periodical is now being received regularly at Institute headquarters. A volunteer reviewer is needed to keep Members informed of pertinent papers in this publication through the "Current Reviews" section of the JOURNAL. Those interested please write the Secretary.

Glidden elected ASCE vice-president

An authority on bridge design and construction, William R. Glidden, bridge engineer for the Virginia State Highway Dept. at Richmond since 1917, has been elected an ASCE vice-president. A graduate of Massachusetts Institute of Technology, Mr. Glidden for many years was on the engineering faculty of the Virginia Mechanics Institute and has taught special classes under the auspices of the Virginia Polytechnic Institute and the University of Virginia. Mr. Glidden has been a member of ACI since 1946 and was a Director in ASCE from 1946-49.

Anderson honored by ASCE

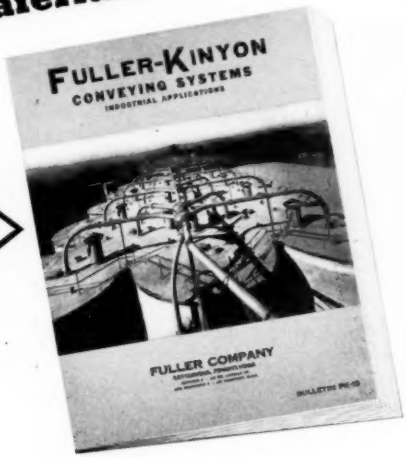
A. A. Anderson was awarded the Arthur M. Wellington Prize by ASCE recently for his paper "Expansion Joint Practice in Highway Construction." He has been with the Portland Cement Assn. for the past 27 years and as manager of the Highways and Municipal Bureau since 1936, he has directed the association's highway work throughout the United States. A graduate of Kansas State College, Mr. Anderson was division engineer of the Kansas State Highway Commission before going to PCA. Affiliated with ACI since 1946, he is author of several technical articles and booklets on concrete pavements.

Fucik elected chairman

E. Montford Fucik, associate of the Harza Engineering Co., Chicago, Ill., was recently elected chairman of the General Education Committee of the Engineering Societies of Chicago, which is composed of delegates from the four Founder Societies and the Western Society of Engineers.

This bulletin is full of ideas how to handle Portland cement and other dry pulverized materials

BULLETIN FK-25



Bulletin FK-25, a 24 page booklet, illustrates and describes the Fuller-Kinyon Conveying System as applied to industry for the unloading and conveying of Portland cement and other materials. It shows, by line drawings and photographs, its application to material handling in asbestos-shingle manufacturing plants, ready-mix concrete plants, phosphate plants, barge and ship unloading, fly-ash handling in power plants, as well as its application in Portland cement manufacturing plants.

Just ask for Bulletin FK-25, and it will be sent you promptly.

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Gillis succeeds Grumm

R. M. Gillis has been appointed California's acting deputy state highway engineer, succeeding Fred J. Grumm who has retired. Mr. Gillis was formerly assistant state highway engineer. A graduate of Whitman College, Wash., he received a CE degree from the Massachusetts Institute of Technology, served with the Washington State Highway Dept. from 1912-26, and was on the staff of the Pacific Bridge Co. until 1929. He joined the California Division of Highways in 1929 as an assistant construction engineer, later becoming a district engineer and then state construction engineer.

Mr. Grumm entered the employ of the highway commission in 1922, becoming successively assistant division engineer, engineer of surveys and plans, assistant state highway engineer, and in 1947, deputy state highway engineer.

Chadwick and Friel named ASCE Directors

W. L. Chadwick, manager of the engineering department of the Southern California Edison Co., Los Angeles, and Francis S. Friel, president and treasurer of the Philadelphia consulting firm of Albright and Friel, Inc., were recently named Directors of ASCE.

Mr. Chadwick, an ACI Member since 1938 and chairman of ACI Committee 805, Recommended Practice for the Application of Mortar by Pneumatic Pressure, has also been active in ASCE activities. Educated at the University of Redlands, he was with the Southern California Edison Co. from 1922-31 and with the Metropolitan Water District of Southern California from 1931-37. Rejoining the Edison Co., he has been manager of the engineering department since 1945. He was also president of the Los Angeles Engineering Council in 1947 and conducted a successful campaign to broaden the California registration law to include other professional engineers.

Mr. Friel, an ACI Member since 1920, has been in consulting practice for 28 years and has directed the engineering staff of Albright and Friel since 1931. An authority in the sewerage field, he has been president of the Federation of Sewage Works Assn. Mr. Friel is a graduate of Drexel Institute.

ASCE confers award on Newmark

An authority in the structural engineering field and an active ACI Member, Nathan M. Newmark, was announced as the winner of the ASCE Moisseiff Award for a paper on "Design of I-Beam Bridges." An ACI Member since 1934, he has been on the University of Illinois engineering staff since 1934 except for a period of war service. He graduated from Rutgers University, and received the degrees of master of science and doctor of philosophy in engineering from the University of Illinois. He is now in charge of the research program in structural engineering in the University of Illinois Engineering Experiment Station. During World War II he was engaged in a number of activities here and abroad through the National Defense Research Committee and the Office of Scientific Research and Development, and later was with the Scientific Advisory Group of the U. S. Air Force. Author of a number of structural papers, Professor Newmark won the ASCE J. James R. Croes Medal in 1944 and ACI's Wason Medal for most meritorious paper for the 1949 paper, with C. P. Siess, "Rational Analysis and Design of Two-Way Concrete Slabs."

Steinman to design Puerto Rican bridge

D. B. Steinman, New York City consultant, has been retained by the government of Puerto Rico to design and supervise the construction of a $1,000,000 bridge over the Martin Pena Channel at San Juan. The structure will be part of the current program of highway construction in the metropolitan area of Puerto Rico, which is being undertaken with joint funds of the insular and federal governments.

Magnel awarded medal

Gustave Magnel, professor of engineering at the University of Ghent, Belgium, recently received the Frank P. Brown Medal of the Franklin Institute for "outstanding development of engineering technique for prestressed concrete."

"The work," the citation continues, "resulted in radical improvements in the design and construction of bridges, buildings and other structures, making it possible to depart from standard concrete arch con-

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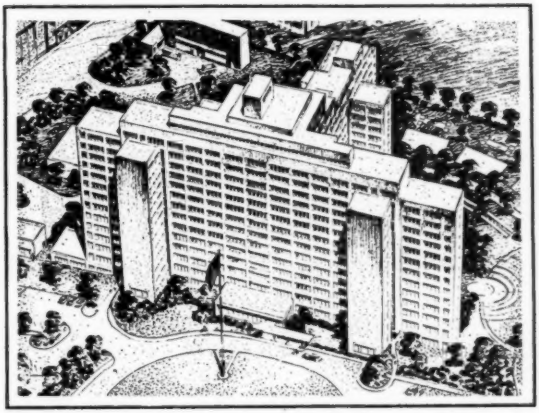

Darex AEA makes possible the use of low (3½") slump in the heavily reinforced concrete specified for the Veterans Administration Hospital, Boston. Scheduled for completion in December, 1951, the project will require 26,000 yards of Darex AEA concrete.

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struction." Professor Magnel's system for design and construction of prestressed concrete was first employed in this country in the construction of the Walnut Lane Bridge in Philadelphia.

Collier appointed senior structural engineer

Theodore F. Collier, previously in charge of structural and architectural layout and design of steam electric and industrial development for Sanderson and Porter of New York City, has joined the staff of Westcott and Mapes, consulting engineers, New Haven, Conn. He will serve the company, which is engaged in power plant and industrial plant design, as senior structural engineer.

Sullivan enters private practice

E. Q. Sullivan, who recently retired as district engineer for the State Division of Highways, San Bernardino, Calif., has opened a consulting office there. Mr. Sullivan served the highway division for 27 years as district engineer where he directed highway construction totaling $75,000,000.

Billig appointed professor

Kurt Billig, for the past 12 years consulting engineer in London, England, has been appointed professor of civil engineering at the University of Hong Kong. Dr. Billig, who has been consultant to the British Ministry of Works, was in the United States last year on a lecture tour reporting on progress made in Europe on prestressed concrete.

Paolella named chairman

Phillip Paolella, vice-president of the Plasticrete Corp., Hamden, Conn., has been named chairman of the Publications and Publicity Committee of the National Concrete Masonry Assn., Chicago, Ill. He succeeds Fred W. Reinhold, president of Anchor Concrete Products, Inc., Buffalo, N. Y.

Rogers awarded WSE prize

Paul Rogers, structural design engineer, Sargent and Lundy, Chicago, Ill., was awarded second place in the recent Western Society of Engineers technical paper contest for his paper "The Structural Aspects of Power Plant Design."

The paper dealt with the special problems of power plant design and Mr. Rogers emphasized the need for knowledge of advanced theory.

Mr. Rogers is also the author of several ACI contributions, one on design of two-way slabs, another on hexagonal bins.

Kray opens office

After 17 years of diversified engineering experience, C. J. Kray, Boston, Mass., has entered private practice as a structural engineer, engaging in design and supervision of construction for structural steel, reinforced concrete and timber structures as well as foundations. He was formerly associated with Fay, Spofford and Thorndike, Consulting Engineers in Boston.

"Research Is Learning"

A comprehensive definition of the role of research in engineering education, compiled by representatives of 83 leading engineering schools, is the central theme of "Research Is Learning," new nontechnical publication of the Engineering College Research Council of the American Society for Engineering Education.

"Engineering research," according to the special report, "has as a basic purpose the discovery of fundamental information as well as the development of specific products useful to mankind.

"The research training of an engineer is training in thinking and reasoning. The excitement and stimulation of truly creative professional work are an indispensable ingredient of the educational environment," the representatives declare.

"Thus," they say, "creative research aids an engineering school to fulfill its educational objective. The same research fulfills a further obligation through the contributions it makes to the wealth of the nation and its people."

Associated with the statement in the 32-page illustrated publication are 12 brief summaries of universities' research activities which are designed to provide specific examples of the more general statements. These cover individual research activities in a wide variety of technical fields, selected from suggestions made by the engineering schools which hold membership in the Research Council.

Draft of engineers poses problem

The deferment of engineers and scientists from military service is an important question of national policy not to be determined lightly or with flippant words, it was declared by Paul H. Robbins, Executive Director of the National Society of Professional Engineers, in commenting upon the remarks of General Lewis B. Hershey, National Selective Service Director, before the Society for Personnel Administration.

"General Hershey's remark that the discussion of the status of engineers and scientists in relation to the total national defense is a 'lot of baloney' indicates a failure to take cognizance of the critical results flowing from our experience in this field during World War II."

"We agree with General Hershey that the total national defense is more important than any individual group," Robbins said. "But in considering that national defense interest, General Hershey is overlooking the role of the professional engineer and scientist."

Robbins suggested that General Hershey re-read the report of the President's Scientific Research Board, commonly known as the Steelman report, issued in 1947, wherein the following comment was made on the policy of drafting science and engineering students:

"In striking a balance among this complex of war pressures, the government decided against deferment of science students. In the light of the effect of the policy in contributing to the present shortage the wisdom of the decision seems dubious."

"Certainly no one is better qualified to discuss this question than Dr. Vannevar Bush, Director of the Office of Scientific Research and Development, during World War II and formerly chairman of the Research and Development Board of the Defense Department," Robbins said. "In his publication, 'Science, The Endless Frontier,' Dr. Bush points out that there was a great deficit of engineers and scientists because of our draft policies in World War II."

NSPE also called attention to the recent remarks of Gordon Dean, Chairman of the Atomic Energy Commission, which supported the viewpoint taken by the leading engineering and scientific societies.

Dean strongly supported training of the technical manpower on whom we so utterly depend for success in waging war and advance in the arts of peace. Scientists and engineers are a national resource of major importance, and it is imperative that we make sensible, effective use of their skills, he said.

Dean stated, "one consideration is to see that we 'keep the seed corn coming' for future growth and development by maintaining in full vigor, year-in-year-out, in wartime as well as in peacetime, the staff for scientific training in the universities. The corollary of this is that we also maintain the flow into the universities of the talented youngsters who can carry scientific and engineering training over to new accomplishment."

"If we fail to provide for continuous training of our technical talent in a long period of mobilization," Robbins went on to say, "who will develop our guided missiles, who will improve upon the atomic weapons, who will have the technical education and experience to bring forth new items such as proximity fuses of the last war?"

"No one argues with General Hershey that the number one job is to supply the armed forces with the fighting manpower it needs, but let us not put these men at the front without the best in equipment and weapons which only America's technical men can conceive and produce," Robbins concluded.

Bridge and structural engineering conference planned

The Fourth Congress of the International Assn. for Bridge and Structural Engineering, to be held in Cambridge, England, in 1952, will feature several papers on concrete and reinforced concrete structures. The topics include the influence of preparation, transport and placing of concrete on the design of structures; tensile strength of concrete; effect of repeated and continuous loading and creep; and corrosion of reinforcement. Current problems in prestressing and dynamic stressing and fatigue strengths are also scheduled for presentation.

The program will also include discussion of design and construction techniques for metal structures and design factors in general, especially safety factors, for all types of structures.

New Members

45 new members were added to ACI's membership list during the month of September to bring the membership total to 5178 on Oct. 1, 1950.

Alabama

Bishop, C. E., (Indiv.) Engineering Dept., Reynolds Alloys Co., Listerhill, Ala.

California

Bleifuss, D. J., (Indiv.) c/o International Engineering Co., Inc., 74 New Montgomery St., San Francisco 5, Calif.
Hansen, Howard J., (Indiv.) U. S. Naval Civil Engineering Research & Evaluation Laboratory, Port Hueneme, Calif.
Pegram, William Wesley, (Indiv.) 304 W. 64th St., Inglewood 3, Calif.
Trask, J. W., (Indiv.) Division of Highways, P. O. Box 311, Redding, Calif.
Weldon, Christopher, (Jr.) APO 187 (HOW), c/o P. M., c/o Holmes & Narver, San Francisco, Calif.

Delaware

Gaylord, Charles Nelson, (Indiv.) Dept. of Civil Engineering, University of Delaware, Newark, Del.

District of Columbia

Fischer, Milton A., (Jr.) 1812 Varnum St., N. E., Washington 18, D. C.
Schwartz, James I., (Jr.) National Brick & Supply Co., Inc., Terra Cotta 11, D. C.

Florida

Higgins, R. H., (Indiv.) 705 Sharar Ave., Opa-locka, Fla.

Illinois

Blechinger, Frank X., (St.) Apt. 625, 1107 W. Green, Urbana, Ill.
Gaston, James R., (St.) 105 Talbot Laboratory, University of Illinois, Urbana, Ill.
Laupa, Armas, (St.) 105 Talbot Laboratory, University of Illinois, Urbana, Ill.
Punyagupta, Somporn, (St.) 504 E. John St., Champaign, Ill.
Riddle, Paul Frederick, (Jr.) 819 Garfield Ave., Aurora, Ill.

Indiana

Curtis, Harry E., (Indiv.) 2001 E. Pontiac St., Fort Wayne, Ind.
DiPirro, Joseph A., (St.) Indiana Technical College, 221 E. Washington, Fort Wayne, Ind.

Iowa

Perez, Jose N. Jr., (St.) N-162 Hillcrest Dorm., Iowa City, Ia.

Kansas

Mann, A. R., (Indiv.) Mann & Co., Archts. & Engrs., P. O. Box 1148, Hutchinson, Kans.

Massachusetts

Bailey, Christopher T., (Indiv.) 25 Whittier Road, Wellesley Hills, Mass.
Brown, George A. L., (Indiv.) One Rollins Place, Boston 14, Mass.

Michigan

Wazir, Azher, (St.) 422 E. Washington St., Ann Arbor, Mich.

New York

Anderson, Frank, (Indiv.) 339 W. 24th St., New York 11, N. Y.
Chu, Kuang-Han, (Indiv.) Ammann & Whitney, 76 Ninth Ave., New York 11, N. Y.
Ditrano, Domenic, (Indiv.) 1576 Unionport Rd., New York 62, N. Y.
Edwards, J. L., (Indiv.) Edwards & Hjorth, 1267 Sixth Ave., New York 19, N. Y.
Kaltenecker, L., (Indiv.) 280 39th St., Lindenhurst, L. I., N. Y.
Wang, Ping-chun, (Indiv.) Ammann & Whitney, Consulting Engrs., 79 9th Ave., New York, N. Y.

Oregon

Howard, John D., (Jr.) Rt. 2, Box 540, Portland, Ore.

Pennsylvania

Eney, William J., (Indiv.) Head of Dept. of Civil Engr. & Mechanics, Lehigh University, Bethlehem, Pa.
Miller, James P., (Indiv.) 124 Market St., Bangor, Pa.

Tennessee

Gibson, Robert E., (Indiv.) Route 13, Fountain City Branch, Knoxville, Tenn.

Texas

Couch, Hampton, (Indiv.) 2310 Moser St., Dallas, Texas
Travis, Lon W., (St.) University of Texas, 221A Brackenridge Hall, Austin 18, Texas

Wisconsin

Vickers, E. G., (Indiv.) Box 263, Waukesha, Wis.

Wyoming

Balog, John Jr., (Jr.) Box 449, Superior, Wyo.
Ferentchak, Frank, (Indiv.) Kemmerer, Wyo.
Moore, Charles F., Jr., (Jr.) 466 N. 9th, Laramie, Wyo.
Williams, Robert Edward, (Jr.) Riverton, Wyo.

Continued on p. 16

Honor Roll

February 1 to September 30, 1950

The ten top names on our Honor Roll have been responsible for sponsoring 107 new applicants since February 1. There are four months remaining on the current Honor Roll and there is still time for **you** to make the "big ten."

Why not introduce the benefits of ACI membership to some business associate who would find participation in Institute activities of interest and value to him in his work?

Newlin D. Morgan (Ill.) **35½**
Newlin D. Morgan, Jr. (Wyo.) **21**
Leon Venegas (La.) **10**
Henry E. Griset (N. C.) **7**
E. I. Fiesenheiser (Ill.) **6½**
Chester P. Siess (Ill.) **6½**
R. H. Sherlock (Mich.) **5½**
A. Amirikian (Md.) **5**
Charles G. Peller (Ind.) **5**
Luis A. Pietri-Lauie (Venezuela) **5**
George C. Alden (Calif.) **4½**
Phil M. Ferguson (Texas) **4**
Henry L. Kennedy (Mass.) **4**
Cesar Oliver-Rugeles (Venezuela) **4**
Ivan M. Viest (Ill.) **4**
Wm. T. F. Hooper, Jr. (Ill.) **3½**
Andrew Reti (Venezuela) **3½**
E. W. Bauman (D. C.) **3**
Earl I. Brown (Ga.) **3**
Raymond E. Davis (Calif.) **3**
Ashby T. Gibbons, Jr. (Ga.) **3**
Emil A. Gramstorff (Mass.) **3**
Walter N. Handy (Ill.) **3**
M. D. Hicklin (Ore.) **3**
James A. McCarthy (Ind.) **3**
Stanley Moore (Canada) **3**
James L. Atkinson (Calif.) **2½**
H. J. Gilkey (Ia.) **2½**
Castor Segundo Goa (Venezuela) **2½**
Orlando W. Irwin (Ill.) **2½**
Frank H. Jackson (D. C.) **2½**
Melvin W. Jackson (Ga.) **2½**
W. H. Jacobs (Ill.) **2½**
S. J. Warberg (Mich.) **2½**
Ralph L. Barbehenn (N. J.) **2**
Roberto Barillas F. (Guatemala) **2**
Sterling Lowe Bugg (Fla.) **2**
W. Fisher Cassie (England) **2**
Clayton M. Crosier (Kans.) **2**
Belmon U. Duval (Ohio) **2**
Eddy N. Hernandez (La.) **2**
Samuel Hobbs (Calif.) **2**
J. E. Jellick (Calif.) **2**
J. Sterling Kinney (N. Y.) **2**
George E. Large (Ohio) **2**
William Lerch (Ill.) **2**
F. J. Ochoa U. (Guatemala) **2**
Rafael Ruiz P. (Guatemala) **2**
Leroy A. Staples (La.) **2**
J. Neils Thompson (Texas) **2**
George D. Youngclaus (Calif.) **2**
J. F. Baxter (N. Y.) **1½**
Oscar Benedetti (Venezuela) **1½**
Anthony Henry Clark (England) **1½**
F. Thomas Collins (Calif.) **1½**
Jacob Creskoff (D. C.) **1½**
Charles W. DeGroff (N. Y.) **1½**
Harlan H. Edwards (Wash.) **1½**
Rodolfo C. Garcia (Philippine Islands) **1½**
Ernst Gruenwald (N. Y.) **1½**
Elmo C. Higginson (Colo.) **1½**
Leo M. Legatski (Mich.) **1½**
Thomas C. Shedd (Ill.) **1½**
John H. Thornton (Scotland) **1½**
A. G. Timms (D. C.) **1½**
Jose Antonio Vila (Cuba) **1½**
Charles E. Wuerpel (Ill.) **1½**
Roderick B. Young (Canada) **1½**
F. O. Anderegg (N. J.) 1
Edward L. Ashton (Ia.) 1
Hugh Barnes (Calif.) 1
Hans H. Bleich (N. Y.) 1
E. F. J. Clark (Canada) 1
Roy R. Clark (Ore.) 1
Kenneth W. Clayton (Australia) 1
Aloysius E. Cooke (Conn.) 1
M. A. Craven (New Zealand) 1
R. A. Crysler (Canada) 1
Walter T. Daniels (D. C.) 1
Harmer E. Davis (Calif.) 1
Raymond E. Davis, Jr. (Calif.) 1
DeWitt Dieterich (Ill.) 1
H. B. Emerson (Ill.) 1
C. C. Everhart (Ia.) 1
Stanley Gordon Fisher (Canada) 1
George W. Ford (Fla.) 1
William E. Gabelman (Calif.) 1
Randall L. Gomien (Ohio) 1
H. F. Gonnerman (Ill.) 1
Otto Graf (Germany) 1
Nathaniel Grant (Mo.) 1
Gilberto Guardia (Panama) 1
Horace P. Hamlin (N. Y.) 1
Robert J. Hansen (Mass.) 1
Terrell R. Harper (Texas) 1
W. L. Harrell (Texas) 1
Charles E. Hawke (Canada) 1
Harry H. Hawley (Ohio) 1
Bernard C. Herring (England) 1
Wallace E. Jobusch (Ind.) 1
Thomas B. Kennedy (Miss.) 1
John C. King (Ohio) 1
K. V. Kini (India) 1
R. A. Kirkpatrick (N. Y.) 1
Blas Lamberti (Venezuela) 1
David V. Lewin (Ohio) 1
Harry E. Lewis (N. Y.) 1
Alvin C. Loewer, Jr. (Pa.) 1
Winthrop E. Luke (Mass.) 1

Charles L. Luther (Ky.).....1
Ian Macallan (New Zealand).....1
Glenway Maxon (Wis.).....1
G. T. McCoy (Calif.).....1
H. J. McGillivray (Fla.).....1
J. H. Murdough (Texas).....1
J. A. Murlin (Texas).....1
James A. Murray (Mass.).....1
J. Neil Mustard (Canada).....1
William D. Nowlin (Va.).....1
Oscar A. Nunez G. (Venezuela).....1
Carlos Ospina (Colombia).....1
Wm. Otcovsky (Ill.).....1
Kenneth K. Paget (Canada).....1
Jose Antonio Pardo B. (Mexico).....1
Dean Peabody, Jr. (Mass.).....1
Howard F. Peckworth (Ill.).....1
Henry A. Pfisterer (Conn.).....1
Stanley A. Phillip (Ill.).....1
Harry W. Piper (Md.).....1
Dan H. Pletta (Va.).....1
Herman G. Protze (Mass.).....1
Frederick A. Reickert (Ill.).....1
Theodore O. Reyhner (Colo.).....1
Evan L. Richard (Australia).....1
Frank E. Richart (Ill.).....1
Horace B. Rickey, Jr. (La.).....1
Bernard L. Robinson (N. Y.).....1
Otto Sauermann (Ill.).....1
Emil Schmid (N. J.).....1
Wilfrid Schnarr (Canada).....1
James E. Schumann (Calif.).....1
Elwyn E. Seelye (N. Y.).....1
Howard Simpson (Mass.).....1
E. R. Smallhorn (Canada).....1
Howard R. Staley (Mass.).....1
M. Eugene Sundt (N. M.).....1
Pascal L. Tasio (N. Y.).....1
Wm. H. Thoman (Colo.).....1
H. F. Thomson (Ill.).....1
E. W. Thorson (Colo.).....1
LeRoy A. Thorssen (Canada).....1
G. E. Troxell (Calif.).....1
G. K. Viall (Wis.).....1
F. E. Votaw (Mass.).....1
A. G. Watt (Pa.).....1
C. A. Willson (N. Y.).....1
S. H. Woodard (N. Y.).....1
A. E. Wynn (S. Africa).....1
Aubrey D. Young (La.).....1
Edward F. Young (Minn.).....1
R. A. Zern (Pa.).....1

The following credits are, in each instance "50-50" with another member.

James Adam, Jr.
J. B. Alexander
Herbert C. Allen
Frederick G. Anderson
Charles D. Babcock
Malcolm F. Baker
C. Merrill Barber
George E. Barnes
Frank Baron
A. Allan Bates
K. P. Billner
G. L. Blanchard
A. E. Bond
Harry E. Born
J. G. Bragg
James E. Branch
Woodrow L. Burgess
R. A. Burmeister
Harold J. Buttery
D. J. Cameron
Julian B. Carson
Jules P. Channing
Ralph E. Coblentz
Miles K. Cooper
Roger H. Corbetta
William A. Crabb
A. E. Cummings
Clayton L. Davis
G. A. DeLong
Harry Delzell
A. H. Douglas
R. G. Douglas, Jr.
Albert W. Dudley
Leonard E. Dunlap
John R. Dwyer
Harry Elksberg
W. J. Emmons
L. G. Farrant
Jorge Figuls
John D. Freeman
P. J. Freeman
Bernardo Fuentes A.
Mario W. Gamalero
J. H. Gandhi
A. H. Garnsey
Grayson Gill
A. T. Goldbeck
O. H. Gosswein
Alex H. Graves
Homer M. Hadley
Albert Haertlein
Theodore F. Hagerman
William H. Hall
W. C. Hansen
Harvey L. Harris
George E. Hatch
John M. Hayes
Philip W. Helsley
W. A. Hohlweg
William N. Holway
Wilfred M. Honour
J. W. Hubler
Virgil L. Hughes
Maurice Hunt
S. E. Hyde, Jr.
Raja A. Iliya
W. R. Ingram
C. E. Ireland
C. L. N. Iyengar
Arthur M. James
M. E. James
J. G. Johnson
Bernerd D. Keatts
J. W. Kelly
Edgar R. Kendall
Frank Kerekes
W. D. Kimmel
W. P. Kinneman
Ralph W. Kluge
Jean H. Knox
M. C. Kolinski
Paul P. Kraai
Richard J. Kroc
J. E. Lothers
Irvin H. Luke
I. D. MacKenzie
M. F. Macnaughton
James V. Mandia
Eastman M. Markell
W. T. McClenahan
Sam McCluer
C. T. J. McDowell
A. J. McElrath
Douglas McHenry
Harmon S. Meissner
Willis T. Moran
V. S. Murray
A. E. Myers
Henry L. Neve
Walter Newland
Roy A. Nyquist
Raymond G. Osborne
John P. Ottesen
Douglas E. Parsons
Thomas Poli
Milos Polivka
Chesley J. Posey
Leo F. Pratt
Walter H. Price
Charles J. Prokop
Raymond C. Reese
Ross M. Riegel
Guy Rinfret
Charles S. Rippon
Richard A. Roberts
A. Rodriguez-Delfino
Pedro Rodriguez
Paul Rogers
Mark R. Rosumny
Oswaldo Rovati B.
William H. Rowan
John A. Ruhling
Frederic Rusche
Nicholas R. Samaha
R. W. Sauer
D. M. Schmid
Axel Schulze
J. Morgan Smith
Myron M. Smith
Ernest L. Spencer
John J. Stedje
Byram W. Steele
John D. Stites
Herbert M. Stoll
Hale Sutherland
Ernest O. Sweetser
L. W. Teller
Walter F. Tews
George P. Thigpen
J. Trueman Thompson
B. K. Thornley, Jr.
T. Thorvaldson
Paul M. Trueblood
Lee Turzillo
Lewis H. Tuthill
Miguel Villa
Julius J. Warner
G. E. Warren
Willard W. Warzyn
W. R. Waugh
L. T. Willoughby
Clement T. Wiskocil
Francis P. Witmer
Bourdette R. Wood
H. D. Worthington

New Members

Continued from p. 14

Colombia

Arias Restrepo, Jaime, (Jr.) Cra. 9a, No. 16-20 of 602, Bogota, Colombia, S. A.

Guatemala

Becker, Otto Ernest, (St.) 12 C. P. No. 10, Guatemala City, Guatemala

Paetáu, Ahax, (St.) 6a Calle Ote de Jocotenango No. 2, Guatemala City, Guatemala

Italy

Lotti, Carlo, (Indiv.) Via del Tritone 132, Rome, Italy

Japan

Takeyama, Kenzaburo, (Indiv.) 2-1644, Nogatamachi, Nakanoku, Tokyo, Japan

Venezuela

Carciente, Jacob, (St.) Los Chaguaramos, Calle Humboldt, Qta. Sol-Mery, Caracas, Venezuela

India to hold engineering exhibition

In conjunction with international engineering conferences to be held in India, January 1951, the Indian Ministry of Works, Mines and Power has announced plans to conduct an International Engineering Exhibition. The exhibition will feature the latest developments in the sciences and techniques covered by the international conference of the Fourth Congress on Large Dams, the sectional meeting of World Power Conferences, the meeting of the International Assn. of Hydraulic Research, and the meeting of the International Commission on Irrigation and Canals.

Statement of the Ownership, Management, and Circulation Required by the Act of Congress of August 24, 1912, as Amended by the Acts of March 3, 1933, and July 2, 1946 (Title 39, United States Code, Section 233)

Of JOURNAL OF THE AMERICAN CONCRETE INSTITUTE published Ten issues a year at Detroit, Michigan, for September, 1950.

1. The names and addresses of the publisher, editor, managing editor, and business managers are: Publisher, American Concrete Institute, 18263 W. McNichols Rd., Detroit 19, Mich.
Editor, Harvey Whipple, 18263 W. McNichols Rd., Detroit 19, Mich.
Managing Editor, None.
Business Manager, None.

2. The owner is: (If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding 1 percent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a partnership or other unincorporated firm, its name and address, as well as that of each individual member, must be given.)
American Concrete Institute, 18263 W. McNichols Rd., Detroit 19, Mich.
Frank H. Jackson, President, U. S. Public Roads Administration, Washington, D. C.
Harry F. Thomson, Vice-President, Materials Service Corporation, Chicago, Ill.
A. T. Goldbeck, Vice-President, National Crushed Stone Association, Washington, D. C.

3. The known bondholders, mortgagees, and other security holders owning or holding 1 percent or more of totalamount of bonds, mortgages, or other securities are: (If there are none, so state.)
NONE.

4. Paragraphs 2 and 3 include, in cases where the stockholder or security holder appears upon the books of the company as trustee or in any other fiduriary relation, the name of the person or corporation for whom such trustee is acting; also the statements in the two paragraphs show the affiant's full knowledge and belief as to the circumstances and conditions under which stockholders and security holders who do not appear upon the books of the company as trustees, hold stock and securities in a capacity other than that of a bona fide owner.

5. The average number of copies of each issue of this publication sold or distributed, through the mails or otherwise, to paid subscribers during the 12 months preceding the date shown above was:.....(This information is required from daily, weekly, semiweekly, and triweekly newspapers only.)

HARVEY WHIPPLE
(Signature of editor)

Sworn to and subscribed before me this 18th day of September, 1950.

JEAN M. STEM, Notary Public
(My commission expires Oct. 17, 1952)

[SEAL]

Italian concrete society organized

A new Italian society for the promotion of concrete construction, *L'Associazione Italiana Cemento Armato*, has been organized in Milan, Italy. Prof. A. Danusso was elected as the organization's first president.

Prestressed railroad trestle slab tested

A prestressed concrete railroad trestle slab, approximately one half the depth of a conventional reinforced concrete slab of the same length and width, withstood a load of 480,000 lb, or about 2.8 times design live load and impact, before failure, in tests recently conducted at the Portland Cement Assn. Research and Development Laboratories, Skokie, Ill.

The full-size trestle slab was 25 x 7 x 1½ ft and designed for an E-72 live load plus full impact. It weighed 20 tons and contained 10 cu yd of concrete with a strength of 7000 psi at 28 days. Prestressing was done with 15 bridge strands 1⅜ in. in diameter, post-tensioned to 120,000 psi.

A strand and socket connection was tested to failure at 228,000 psi with no evidence of failure of the end connection. The strand was composed of 37 5-mm high-tensile steel wires encased in plastic tubing to prevent bonding to the concrete.

The prestressed slab was loaded to full design load. The load was then increased until the first hairline crack appeared at approximately 1½ times live load and impact. Upon removal of the load the crack disappeared entirely, indicating the complete elasticity of the slab. Finally the slab was loaded until failure occurred in the upper portion at approximately 2.8 times design load.

Impact on highway bridges studied

An analytical investigation of impact on highway bridges begun recently at the Engineering Experiment Station of the University of Illinois and under the direction of Dr. N. M. Newmark, research professor of structural engineering, will provide a better theoretical basis for the planning of field tests and interpretation of results from such tests. Cooperating agencies are the Illinois Division of Highways and the Bureau of Public Roads.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Combined form and reinforcement

Two leaflets have been issued by Granco Steel Products Co. describing "Cofar," a reinforced concrete construction using high-strength, deep-corrugated galvanized steel for both reinforcement and form. Welded cross wires tie the corrugated steel and concrete together. The manufacturer claims that Cofar is capable of resisting high reinforcement stresses in addition to full form loads and replaces conventional reinforcing bars and temperature bars at the bottom of the concrete slab. Tables of permissible loads and properties are included in the leaflets.

Copies of the leaflets are available from Granco Steel Products Co., Granite City, Ill.

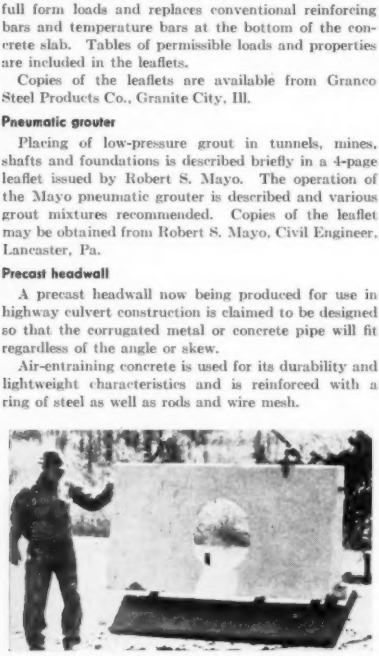
Pneumatic grouter

Placing of low-pressure grout in tunnels, mines, shafts and foundations is described briefly in a 4-page leaflet issued by Robert S. Mayo. The operation of the Mayo pneumatic grouter is described and various grout mixtures recommended. Copies of the leaflet may be obtained from Robert S. Mayo, Civil Engineer, Lancaster, Pa.

Precast headwall

A precast headwall now being produced for use in highway culvert construction is claimed to be designed so that the corrugated metal or concrete pipe will fit regardless of the angle or skew.

Air-entraining concrete is used for its durability and lightweight characteristics and is reinforced with a ring of steel as well as rods and wire mesh.



The headwalls can be installed with or without foundation base and wing walls.

Further information may be secured from Protex Industries, 1331 West Evans, P. O. Box 4044, South Denver Station, Denver, Colo.

Perlite

Two reports recently published by the Perlite Institute supply comprehensive basic information on perlite plaster and concrete, including mix proportions in plaster, k factors and strength of concrete in several densities. One report also includes 17 detailed fire resistance ratings by A.S.T.M. standard methods. The reports are available from the Perlite Institute, 35 W. 53rd Street, New York 19, N. Y.

New air-entraining agent

Tests have indicated that sodium parachlorobenzenesulfonate monohydrate may have application in the air entrainment field. The sodium salt is a free flowing, coarsely crystalline, water soluble compound which has a molecular weight of 233 and is thermally stable except for loss of its water up to 300 C. A bulletin describing the physical properties and use of the compound will be sent upon request to the Market Research Division, Pennsylvania Salt Mfg. Co., 1000 Widener Building, Philadelphia 7, Pa.

Fireproofing details

Fireproofing with lightweight Permalite plaster (perlite) is described in two folders released by Great Lakes Carbon Corp. They include detailed drawings and a short form specification which can be used as part of a job specification. One of the folders is on the fireproofing of steel columns and gives specifications for getting ratings of 1, 2, 3 or 4 hours. The other folder describes a suspended ceiling under non-combustible construction which has a 4-hour fire rating. The folders are available on request from Great Lakes Carbon Corp., Building Products Division, 18 East 48th Street, New York 17, N. Y.

Torque converter truck mixer transmission

A new development in truck concrete mixer transmissions incorporates a torque converter and fluid flywheel. Advantages claimed by the manufacturer are a 50 percent reduction in required horsepower, absence of shock loads and increased clutch life. In the past, 4½- and 5-cu yd truck mixers have used an engine of approximately 55 hp; using the torque converter transmission the manufacturer states that the engine can be reduced to 28 hp. The Transo transmission has a 3 to 1 power multiplying factor. Further information may be obtained from the Transmission and Gear Co., 10421 Haggerty Ave., Dearborn, Mich.

Concrete mixers

A booklet by Worthington Pump and Machinery Corp. illustrates how big "Blue Brute" concrete mixers, when installed in a central plant, can be used in conjunction with truck mixers or agitators. Attachments and accessories, such as the hydraulic control system, are illustrated and complete specifications and dimensions of four mixers are given, a 1-yd portable or stationary, 2-yd stationary, 3-yd stationary and a 4½-yd stationary model. Bulletin B-1700-B2 is available from Worthington Pump and Machinery Corp., Ransome Construction Equipment Sales Division, Dunellen, N. J.

Fiberglass expansion joint

An asphalt impregnated, bonded glass wool board has been developed by Owens-Corning Fiberglas Corp. for use as an expansion joint filler. The manufacturer states that when the new filler is compressed to 50 percent of its thickness, more than 70 percent of its original thickness can be recovered. It is claimed

that the durability and longer life of Fiberglas expansion joint are due to the bonding of glass fibers with resins and blended asphalts. Copies of a 4-page bulletin describing the new joint filler are available from Owens-Corning Fiberglas Corp., Toledo 1, Ohio.

Nonagitating truck body

Several new bulletins released by Maxon Construction Co., describe the use of Dumpcrete, a nonagitating concrete truck body, on highway projects, mass construction and small jobs. Well illustrated, the bulletins also show typical plant installations as used on various construction projects. The Dumpcrete body can also be converted for rock or earth moving by removing the baffle and chute assembly. The bulletins can be obtained from Dumpcrete Division, Maxon Construction Co., Inc. 131 N. Ludlow St., Dayton 2, Ohio.

Earthquake resistant design

A revised edition of "Earthquake Resistant Design of Buildings, Structures and Tank Towers" has been published by the Pacific Fire Rating Bureau. The 84-page booklet includes design recommendations for lateral force, as well as recommendations on material for and construction of earthquake resistant structures.

A specification is given for tank supporting structures and towers and examples of tank tower analysis of concrete and steel, are included.

The final section of the booklet illustrates structural damage suffered in various earthquakes from 1906 to 1949.

Precast joist machine

The "Dunbeam" precast joist machine is described in a recent brochure from W. E. Dunn Mfg. Co. The machine is a continuous production machine for making precast concrete joists of 6, 8, 10, 12, 14 and 16 in. heights, slabs up to 24 in. wide, and of any practical length.

The brochure can be obtained from W. E. Dunn Mfg. Co., Holland, Mich.

SR-4 strain recorder

A new Baldwin strip chart strain recorder for continuous measurement of surface strain in structures by SR-4 strain gages has been announced. The recorder is an electronic type instrument for use with two SR-4 gages having a resistance of 120 ± 1.2 ohms and a sensitivity factor between 1.9 and 2.2.

Usually one gage is active and the other used for temperature compensation but both may be active for measuring combined bending and tensile stresses or differential strains.

Further information is available from Baldwin Locomotive Works, Philadelphia 42, Pa.

Corrosion-proof cements

The Atlas Mineral Products Co. has published a new bulletin, No. 5-1, on "corrosion-proof cements." The bulletin describes Atlas' complete line of resin, sulfur, silicate and asphaltic cements. Information on each cement is compiled in handy chart form.

Copies are available from Atlas Mineral Products Co., 43 Walnut St., Mertztown, Pa.

Reinforcing bar card

A new "Bar Card," covering latest ASTM Specification A-305 for reinforcing bars, has been published by the Concrete Reinforcing Steel Institute. The card serves as a ready reference chart of weights (lb per ft) and nominal dimensions (diameter, cross-sectional area and perimeter) for standard sizes of steel reinforcing bar from ¼ thru 1¼ in. Bar sizes are listed in both the old system (in.) and the new (numbered) system, recently recommended by the U. S. Dept. of Commerce.

Listed on the back of the card are standard ASTM designation numbers for reinforcing bars, including those for "grades of steel" and "deformed bars."

Copies of the new card can be obtained by writing the Concrete Reinforcing Steel Institute, 38 S. Dearborn St., Chicago 3, Ill.

Lightweight aggregate

A new illustrated 50-page Permalite job data brochure has been released by Great Lakes Carbon Corp It contains typical Permalite jobs with such job data as name of architect, general contractor, sub-contractor and the amount of Permalite and how it was used.

The brochure is available from Great Lakes Carbon Corp., Building Products Division, 18 East 48th St., New York 17, N. Y., or 612 S. Flower St., Los Angeles 17, Calif.

Long span roofs

"Long Span Roof Structures," a reprint from *Architectural Forum*, has been released by the consulting firm of Ammann and Whitney. The booklet, well-illustrated, describes long span roof designs for auditoriums, sports arenas, storage buildings and aircraft hangars.

Copies of the booklet are available from Ammann and Whitney, Consulting Engineers, 76 Ninth Ave., New York 11, N. Y., or 724 E. Mason St., Milwaukee 2, Wis.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 22 are currently available at prices indicated. Please order by title and title number.

SOME OBSERVATIONS ON THE USE OF REINFORCING STEEL IN CONCRETE PAVEMENTS.....47-1

Price 35 cents.

BENGT F. FRIBERG—Sept. 1950, pp. 1-16 (V. 47)

The paper contains a few accumulated observations from a review of literature on the evolution of reinforced concrete pavement designs, with reference to various systems of reinforcement which have seen extended use. Typical structural concrete pavement failures are described. Trends in design and reinforcement practice are shown, with special reference to recent developments. Pertinent findings of a few published pavement surveys are cited, with special attention to the 900-mile Louisiana pavement survey made in 1945. The need for performance information and additional research is stressed.

TESTS OF PAPER MOLDS FOR CONCRETE CYLINDERS.....47-2

Price 35 cents.

ROBERT A. BURMEISTER—Sept. 1950, pp. 17-24 (V. 47)

Concrete test cylinders cast in a new type paper mold had a compressive strength lower than that predicted for the concrete mix used. Investigation showed that cracks and mechanical injuries to the outer shell of the concrete cylinder caused by movement of the paper stock during the first 24 hours of curing reduced the strength of the cylinder. To a lesser degree this was true also of the paraffined paper molds in common use for casting test cylinders, specimens cast in both types of paper molds showing lower strengths than test cylinders from the same mix cast in steel molds.

ADMIXTURES IN CONCRETE.....47-3

Price 60 cents.

W. T. MORAN, F. H. JACKSON, BRUCE E. FOSTER and T. C. POWERS—Sept. 1950, pp. 25-52 (V. 47)

Five papers by members of ACI Committee 212, Admixtures, are, because of their common general subject, presented together.

Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.

The advantages and disadvantages of several types of admixtures used in the fabrication of various concrete products, such as building block, cast stone, pipe, cribbing and curbing, are discussed. The admixtures considered are classified into the following groups: accelerators, air-entraining agents, gas-forming agents, water repellent agents, and workability agents.

The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.

Present knowledge of admixtures in counteracting alkali-aggregate reactions is reviewed. It is emphasized that further studies may revise thinking in this field.

The effectiveness of various concrete admixtures in inhibiting the capillary flow of water and the flow of water under pressure is considered. The types of admixtures included in the discussion are accelerators, soaps, butyl stearate, finely subdivided dry materials, mineral oil, workability agents, and a miscellaneous group of proprietary compounds.

IMPROVED SONIC APPARATUS FOR DETERMINING THE DYNAMIC MODULUS OF CONCRETE SPECIMENS.....47-4

Price 35 cents.

C. E. GOODELL—Sept. 1950, pp. 53-60 (V. 47)

After a brief introduction to sonic testing and a description of commercial equipment, the apparatus built for the Michigan State Highway Department is discussed. Reliable results can be obtained by an unskilled worker with this compact equipment which has twice the frequency band spread of the usual oscillator. A wiring diagram of the sonic apparatus is included.

ANALYSIS OF THREE-DIMENSIONAL BEAM-AND-GIRDER FRAMING.47-5

Price 35 cents.

PHIL M. FERGUSON—Sept. 1950, pp. 61-72 (V. 47)

The beam-and-girder floor with some beams carried directly by columns and others supported on girders is cited as a practical problem in frame analysis that must include the torsional stiffness of the girder. Curves showing how moment coefficients vary with this torsional stiffness are developed for a few simple cases in interior panels. These show the weakness of rule-of-thumb methods.

A practical calculation form is set up for use with the moment distribution method in solving three-dimensional problems of this type.

Proposed Revision of SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (617-44).....47-6

Price 35 cents.

REPORT OF COMMITTEE 617—Oct. 1950, pp. 93-116 (V. 47)

New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

LINEAR TRAVERSE TECHNIQUE FOR MEASUREMENT OF AIR IN HARDENED CONCRETE.....47-7

Price 35 cents.

L. S. BROWN and C. U. PIERSON—Oct. 1950, pp. 117-124 (V. 47)

The method described for the determination of air in hardened concrete permits the examination of 6 x 8-in. and 6 x 10-in. random plane face-ground hardened concrete specimens which more truly represent the aggregate and air voids in the actual concrete than smaller specimens. The construction and use of the instruments are discussed and results of tests are given. Because of the time and equipment necessary to measure air content by means of the integrator, it is not adaptable to field use. However, as a laboratory tool it provides a means for quick and accurate determination of total air.

INFLUENCE OF THE QUALITY OF MORTAR AND CONCRETE UPON CORROSION OF REINFORCEMENT.47-8

Price 35 cents.

RACHEL FRIEDLAND—Oct. 1950, pp. 125-140 (V. 47)

In tests to determine the influence of the quality of mortar and concrete upon corrosion of reinforcement the variables studied were cement content, water-cement ratio, consistency, grading and depth of cover. The specimens, stored in moist air or exposed to weather, were tested up to the age of 2 years.

The results indicate that consistency has a pronounced effect upon the protective value of mortar and concrete, and that there appears to exist an "optimum consistency" at which the quantity of rust is practically unaffected by time. It was also found that the usual cement contents in reinforced concrete have only a limited effect upon corrosion. It is concluded that water-cement ratio does not in itself control the rate of corrosion of reinforcement.

PROPER SAND GRADING IMPROVES MASS CONCRETE.....47-9

Price 35 cents.

THOMAS B. KENNEDY—Oct. 1950, pp. 141-152 (V. 47)

Two series of concrete mixtures were designed using 6-in. traprock coarse aggregate and eight separate gradings of natural sand ranging in fineness modulus from 3.60 to 1.35. Tests were made of the plastic concrete, and specimens were cast for tests of compressive strength, resistance to freezing and thawing, and drying shrinkage. Both series of concrete mixtures had a cement content of 2.5 bags per cu yd; one had a normal air content—4½ ± 1½ percent in the portion of the mixture passing the 1½-in. sieve—and the other had a high air content—10 ± 2 percent in the portion of the mixture passing the 1½-in. sieve. Tests indicate that good durability in freezing and thawing can be obtained within the normal air content range with fineness modulus between 2.50 and 2.90. With increased air content, however, the fineness modulus range can be increased to extend from 1.58 to 3.24. Compressive strength was generally affected adversely by increased air content, but not to a serious degree. Drying shrinkage was less with normal air content mixes than with high air content mixtures. It was least when a fineness modulus of 2.52 was used, little difference being apparent between the high and normal air content mixes with this fineness modulus. The air-entraining admixture requirement increased greatly as the fineness modulus of the sand decreased. The water ratio also tended to increase with decreasing fineness modulus.

WATER-SOLUBILITY OF ALKALIES IN PORTLAND CEMENT.....47-10

Price 35 cents.

J. L. GILLILAND and T. R. BARTLEY—Oct. 1950, pp. 153-160 (V. 47)

In an effort to show correlation of soluble alkalies with alkali-aggregate reaction, the authors hydrated a number of cements for periods up to 90 days and analyzed water extracts of the ground hydrated cement. However, the correlation with expansions of mortar bars prepared with reactive aggregate was not improved by considering water-soluble alkalies rather than total alkalies.

The rate at which the alkalies become water-soluble in hydrating cement indicates that the alkali-bearing phases in cement hydrate quite readily.

NEW PRESTRESSING METHOD UTILIZES VACUUM PROCESS.....47-11

Price 35 cents.

K. P. BILLNER—Oct. 1950, pp. 161-176 (V. 47)

The method outlined here was developed to simplify prestressing of concrete to make it generally adaptable to American ways of construction. It eliminates costly anchorages; uses large diameter wires (⅝-in. diameter now available on the market), instead of the customary ⅛-in. diameter wire, thus greatly reducing the number of wires required; prestresses all the wires in the building element simultaneously and simplifies forming. A simplified method of design calculations for prestressed concrete and the result of tests of a beam so designed are included.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE 47-12

Price 35 cents.

REPORT OF COMMITTEE 805—Nov. 1950, pp. 185-196 (V. 47)

This proposed ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

DETERMINING OPTIMUM CROSS SECTIONS FOR PRESTRESSED CONCRETE GIRDERS.....47-13

Price 35 cents.

FRED J. UZIEL—Nov. 1950, pp. 197-212 (V. 47)

General solutions are presented for selecting economically optimum cross sections for prestressed concrete flexural members. Such a solution for simply-supported girders of rectangular cross section, for any span and load, assuming the wires prestressed after setting of the concrete and full dead weight to act during the prestressing operation, leads to a design procedure which, in the case considered, is extremely simple and rapid to apply. It also permits comparisons and studies related to the critical cross section to be made in a more general and conclusive manner. For cross sections other than rectangular, the number of variables making the solution more indeterminate in nature, a similar direct solution is not obtainable. However, a rapid way of obtaining the minimum areas of concrete and steel required is suggested.

STEAM CURING PROTECTS WINTER CONCRETING.....47-14

Price 35 cents.

C. O. CRANE—Nov. 1950, pp. 213-216 (V. 47)

The use of live steam for protecting newly placed concrete from freezing weather and for providing initial curing has resulted in excellent concrete in the Enders Dam spillway at no greater cost than less desirable dry heating methods. This brief paper describes in detail the methods used.

ANALYSIS OF CONTINUOUS CIRCULAR CURVED BEAMS.....47-15

Price 35 cents.

BECLA VELUTINI—Nov. 1950, pp. 217-228 (V. 47)

Continuous circular curved beams can be analyzed easily by the moment distribution method if both bending and torsional end couples are considered. Formulas and tables are presented for circular curved beams of constant cross section that give the relations between the end moments and end torques and the corresponding rotations of the end sections. A proposed method of procedure is illustrated in which the bending end couples are kept separate from the torsional couples. The mathematical operations are not difficult as the convergence is rapid, but attention must be paid to the sign convention which must be definite and easy to apply.

SHEAR RESISTANCE OF TILE-CONCRETE FLOOR JOISTS.....47-16

Price 35 cents.

J. NEILS THOMPSON and PHIL M. FERGUSON—Nov. 1950, pp. 229-236 (V. 47)

Tests on certain types of tile-concrete joists indicate that the tile webs are more effective in resisting diagonal tension than is indicated by the current ACI Building Code specification. Stagger of tile joints appears to be unnecessary, since they do not seem to be planes of weakness insofar as diagonal tension is concerned. The tile reduces the deflection of the joist.

SOLUTION OF DIFFICULT STRUCTURAL PROBLEMS BY FINITE DIFFERENCES.....47-17

Price 35 cents.

ALFRED PARME—Nov. 1950, pp. 237-256 (V. 47)

Finite differences can be applied to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially, the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations necessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values. The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

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DISCUSSION

Discussion closes January 1, 1951

Sept. Jl. '50

Some Observations on the Use of Reinforcing Steel in Concrete Pavements—Bengt F. Friberg

Tests of Paper Molds for Concrete Cylinders—Robert A. Burmeister

Admixtures in Concrete—W. T. Moran, F. H. Jackson, Bruce E. Foster and T. C. Powers

Improved Sonic Apparatus for Determining the Dynamic Modulus of Concrete Specimens—C. E. Goodell

Analysis of Three-Dimensional Beam-and-Girder Framing—Phil M. Ferguson

Discussion closes February 1, 1951

Oct. Jl. '50

Proposed Revision of Specifications for Concrete Pavements and Bases—Committee 617

Linear Traverse Technique for Measurement of Air in Hardened Concrete—L. S. Brown and C. U. Pierson

Influence of the Quality of Mortar and Concrete upon Corrosion of Reinforcement—Rachel Friedland

Proper Sand Grading Improves Mass Concrete—Thomas B. Kennedy

Water-Solubility of Alkalies in Portland Cement—J. L. Gilliland and T. R. Bartley

New Prestressing Method Utilizes Vacuum Process—K. P. Billner

Discussion closes March 1, 1951

Nov. Jl. '50

Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure—Committee 805

Determining Optimum Cross Sections for Prestressed Concrete Girders—Fred Uziel

Steam Curing Protects Winter Concreting—C. O. Crane

Analysis of Continuous Circular Curved Beams—Becla Velutini

Shear Resistance of Tile-Concrete Floor Joists—J. Neils Thompson and Phil M. Ferguson

Solution of Difficult Structural Problems by Finite Differences—Alfred Parme

Proposed changes in the ACI Building Code allow for the improved properties of new-style deformed bars complying with ASTM Specification A 305-49.

Proposed Revision of

Building Code Requirements for Reinforced Concrete (ACI 318-47)*

Reported by ACI Committee 318

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SYNOPSIS

Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special anchorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

EDITORIAL NOTE

Except for section titles, unchanged text in sections of the Building Code in which changes are proposed appears in light face type. Language of the Code which it is proposed to *change or delete* is shown in italics followed by the new wording in bold face type. Both the present and proposed versions of Table 305(a)—Allowable Unit Stresses in Concrete, are given in their entirety to avoid the possibility of confusion.

*Title No. 47-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

This report was completed under the chairmanship of Mr. Post prior to his death. It was submitted to letter ballot of the committee which consists of 28 members; 24 members returned their ballots of whom 20 have voted affirmatively, 3 approve it in part, 1 did not vote and none voted negatively. It is released by the Standards Committee for publication and discussion with view to its consideration for adoption at the 47th Annual Convention, San Francisco, Feb. 20-22, 1951.

†Deceased.

‡Appointed chairman by the ACI Board of Direction, Oct. 23, 1950.

104-Definitions

* * *

Deformed bar—Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling. **A reinforcing bar conforming to the "Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement" (ASTM Designation: A 305). Bars not conforming to these specifications shall be classed as plain bars.** Wire mesh with welded intersections not farther apart than *twelve* **6** in. in the direction of the principal reinforcement and with cross wires not smaller than No. 10 W. & M. gage may be rated as a deformed bar.

* * *

105—ASTM specifications cited in this code

After A 185-37 add:

A 305-49 Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement.

201—Metal reinforcement

Add a new sentence to subsection (a):

Deformations on deformed bars shall conform to "Standard Specifications for Minimum Requirements for the Deformations of Deformed Bars for Concrete Reinforcement" (ASTM Designation: A 305).

505—Placing reinforcement

(*a*) Metal reinforcement shall be accurately placed and adequately secured in position by concrete or metal chairs or spacers. *The minimum clear distance between parallel bars shall be one and one-half times the diameter for round bars and twice the side dimension for square bars. If special anchorage as required in Section 903 is provided, the minimum clear distance between parallel bars shall be equal to the diameter for round bars and one and one-half times the side dimension for square bars.* **The minimum clear distance between parallel bars, except in columns, shall be equal to the nominal diameter of the bars.** In no case shall the clear distance between bars be less than 1 in., nor less than one and one-third times the maximum size of the coarse aggregate. Where reinforcement in beams or girders is placed in two or more layers, the clear distance between layers shall be not less than 1 in., and the bars in the upper layers shall be placed directly above those in the bottom layer.

902—*Ordinary* anchorage requirements

(*a*) Tensile negative reinforcement in any span of a continuous, restrained or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar, whether required for positive or negative reinforcement, shall be extended at least twelve diameters beyond the point at which it is no longer needed to resist stress. *In cases*

where the length from the point of maximum tensile stress in the bar to the end of the bar is not sufficient to develop this maximum stress by bond alone, the bar shall be extended to such a point that with the addition of a standard hook (see Section 906(c)), the maximum tensile unit stress can be developed. **The maximum tension in any bar must be developed by bond on a sufficient straight or bent embedment or by other anchorage (see Section 906).** If preferred, the bar may be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the reinforcement which resists moment of opposite sign.

(*b*) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of *ten or more bar diameters* **6 in.** *Where extension of the reinforcement into the support a distance of ten or more bar diameters is impracticable the bars shall be extended as far as possible into the support and terminated in standard hooks or other adequate anchorage.*

(*c*) *In simple beams, or at the outer or freely supported ends of end spans of continuous beams, at least one-half the positive reinforcement shall extend along the same face of the beam into the support a distance of twelve or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.*

(*c*) In simple beams, or at the freely supported end of continuous beams, at least one-third the required positive reinforcement shall extend along the same face of the beam into the support a distance of 6 in.

903—Special anchorage requirements

(*a*) *Where increased shearing or bond stresses are permitted because of the use of special anchorage (see Section 305), every bar except those specifically mentioned in Section 902(b), shall be terminated in a standard hook in a region of compression, or shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the reinforcement resisting moment of opposite sign.*

903—Plain bars in tension

Plain bars in tension shall terminate in standard hooks except that hooks shall not be required on the positive reinforcement at interior supports of continuous members.

904—Anchorage of web reinforcement

(*a*) Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

1. Welding to longitudinal reinforcement,

2. Hooking tightly around the longitudinal reinforcement through 180 degrees.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE—PRESENT CODE

Description		Allowable unit stresses				
		For any strength of concrete in accordance with Section 302 $n = \frac{30000}{f'_c}$	For strength of concrete shown below			
			f'_c = 2000 psi n = 15	f'_c = 2500 psi n = 12	f'_c = 3000 psi n = 10	f'_c = 3750 psi n = 8
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.45f'_c$	900	1125	1350	1688
Extreme fiber stress in tension in plain concrete footings	f_c	$0.03f'_c$	60	75	90	113
Shear: v (as a measure of diagonal tension)						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$	60	75	90	113
**Footings	v_c	$0.03f'_c$ but not to exceed 75 psi	60	75	75	75
Bond: u						
In beams and slabs:						
Plain Bars	u	$0.04f'_c$ but not to exceed 160 psi	80	100	120	150
Deformed Bars	u	$0.05f'_c$ but not to exceed 200 psi	100	125	150	188
In beams and slabs and one-way footings:						
Plain bars (hooked)	u	$0.06f'_c$ but not to exceed 200 psi	120	150	180	200
Deformed bars (hooked)	u	$0.075f'_c$ but not to exceed 250 psi	150	188	225	250
In two-way footings:						
Plain bars (hooked)	u	$0.045f'_c$ but not to exceed 160 psi	90	113	135	160
Deformed bars (hooked)	u	$0.056f'_c$ but not to exceed 200 psi	112	140	168	200
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area or less†	f_c	$0.375f'_c$	750	938	1125	1405

*See Section 807. **See Section 905(a) and 808(a).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE—PROPOSED REVISION

Description		Allowable unit stresses					
		For any strength of concrete in accordance with Section 302 $n = \frac{30{,}000}{f'_c}$	Maximum value, psi	For strength of concrete shown below			
				f'_c = 2000 psi n = 15	f'_c = 2500 psi n = 12	f'_c = 3000 psi n = 10	f'_c = 3750 psi n = 8
Flexure: f_c							
Extreme fiber stress in compression	f_c	$0.45f'_c$		900	1125	1350	1688
Extreme fiber stress in tension in plain concrete footings	f_c	$0.03f'_c$		60	75	90	113
Shear: v (as a measure of diagonal tension)							
Beams with no web reinforcement	v_c	$0.03f'_c$		60	75	90	113
Beams with properly designed web reinforcement	v	$0.12f'_c$		240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$		60	75	90	113
**Footings	v_c	$0.03f'_c$	75	60	75	75	75
Bond: u							
Deformed bars							
Top bars†		$0.07f'_c$	245	140	175	210	245
In 2-way footings (except top bars)		$0.08f'_c$	280	160	200	240	280
All others		$0.10f'_c$	350	200	250	300	350
Plain bars (must be hooked)							
Top bars		$0.03f'_c$	105	60	75	90	105
In 2-way footings (except top bars)		$0.036f'_c$	126	72	90	108	126
All others		$0.045f'_c$	158	90	113	135	158
Bearing: f_c							
On full area	f_c	$0.25f'_c$		500	625	750	938
On one-third area or less‡	f_c	$0.375f'_c$		750	938	1125	1405

*See Section 807. **See Sections 905(*a*) and 808(*a*).
†Top bars are horizontal bars so placed that more than 12 in. of concrete is cast in the member below the bar.
‡The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

3. Embedment above or below the mid-depth of the beam on the compression side, a distance sufficient to develop the stress to which the bar will be subjected at a bond stress of not to exceed *.04* **0.045**f'_c on plain bars nor *.05* **0.10** f'_c on deformed bars.

* * *

(*e*) Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by one of the following methods:

* * *

3. By bond, at a unit bond stress not exceeding *.04* **0.045** f'_c on plain bars nor *.05* **0.10** f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, plus an extension of the bar, parallel to the upper or lower surface of the beam, of not less than two times the diameter of the bar, plus an extension of the bar, parallel to the upper or lower surface of the beam, of not less than 12 diameters of the bar terminating in a standard hook. This short radius bend extension and hook shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 psi.

4. By bond, at a unit bond stress not exceeding *.04* **0.045** f'_c on plain bars nor *.05* **0.10** f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam and continuous with the longitudinal reinforcement. The short radius bend and continuity shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 psi.

* * *

905—Anchorage of bars in footing slabs

(*a*) *All* **plain** bars in footing slabs shall be anchored by means of standard hooks. The outer faces of these hooks **and the ends of deformed bars** shall be not less than 3 in. nor more than 6 in. from the face of the footing.

1103—Spirally reinforced columns

* * *

(*c*) *Splices in vertical reinforcement*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows:

1. For deformed bars—with concrete having a strength of 3000 psi or above, twenty-four diameters of bar of intermediate grade steel and thirty diameters of bar of hard grade steel. For bars of higher yield point, the amount of lap shall be increased in proportion to the nominal allowable stress. When the concrete strengths are less than 3000 psi, the amount of lap shall be one-third greater than the values given above.

1. For deformed bars with concrete having a strength of 3000 psi or above, 20 diameters of bar of intermediate or hard grade steel. For bars of higher yield point, the amount of lap shall be increased

one diameter for each 1000 psi by which the allowable stress exceeds 20,000 psi. When the concrete strengths are less than 3000 psi, the amount of lap shall be one-third greater than the values given above.

2. For plain bars—the minimum amount of lap shall be *twenty-five percent greater than* **twice** that specified for deformed bars.

3. Welded splices or other positive connections may be used instead of lapped splices. Welded splices shall preferably be used in cases where the bar diameter exceeds *1¼*, **1½** in. An approved welded splice shall be defined as one in which the bars are butted and welded and that will develop in tension at least the yield point stress of the reinforcing steel used.

* * *

Title No. 47-19

Michigan tests indicate the place of membrane curing in the pavement construction scheme.

Curing Concrete Pavements With Membranes*

By C. C. RHODES†

SYNOPSIS

To provide data to assess the advisability of continuing membrane curing of concrete pavements as an alternate method, laboratory and field tests were made to compare the effect of storage conditions on the physical properties of concrete, warping, temperature control, and strength and abrasion resistance of concrete cured with membranes and with wet burlap. A survey of pavements cured with clear membranes in spring and summer showed that cracking, when it occurred at all, was found predominantly in pavements laid in the morning hours. Comparing white-pigmented membranes with the usual wet-curing conditions in the field it was found to be efficient, practicable and about half as expensive as wet curing under the same conditions.

Despite the obvious advantages of membrane curing, many engineers have considered it only a wartime substitute for wet curing to be abandoned as soon as conditions permitted. Thus a thorough evaluation of the membrane method was desirable to determine whether it should be continued as an alternate method of curing concrete pavements.

Too little attention has been devoted in the past to an important function of curing—that of stress control by regulating early temperature and moisture changes in the concrete slab. Throughout this investigation, therefore, this regulatory function received fully as much emphasis as the hydration needs of the cement. Discussion of the fundamental factors in curing is contained in the basic report[1] of which this paper is a condensation.

The results indicate that membrane curing is an acceptable method provided the concrete is protected from the radiant heat of the sun during hot, clear weather to reduce premature cracking to a minimum. When employing membranes, probably the use of white pigments in the curing compound is the most practicable means of temperature control since it involves only a single field operation and is immediately active in retarding temperature rise.

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TABLE 1—SUMMARY OF CABINET

Curing conditions			8 hours			16 hours				24 hours			
Temp., F	Rel. hum., percent	Seal, sq ft per gal.	Moist. loss, percent	Dyn. mod., 10^6psi	Flex. str., psi	Moist. loss, percent	Length change, percent	Dyn. mod., 10^6psi	Flex. str., psi	Moist. loss, percent	Length change, percent	Dyn. mod., 10^6psi	Flex. str., psi
75	30	0	12.1	.27	7	28.6	−.007	1.58	59	49.7	−.009	3.42	146
		200	10.0	.35	12	14.3	+.002	2.39	117	19.7	−.002	4.28	257
		100	11.5	.29	14	14.7	+.006	2.59	114	17.2	+.004	4.34	275
	50	0	10.1	.56	21	30.3	+.002	1.91	104	43.3	+.001	3.15	153
		200	6.6	.42	25	14.8	+.005	2.36	154	21.2	+.005	4.09	252
		100	6.8	.42	29	12.8	+.005	2.47	170	14.9	+.005	4.32	309
	95	0	8.7	.35	22	13.6	.000	2.43	116	17.9	+.001	4.58	276
		200	8.2	.35	22	12.6	+.004	2.58	120	14.5	+.003	5.32	331
		100	7.5	.38	21	11.8	+.003	2.62	150	12.8	+.002	5.26	346
	in water*	0	11.3	.47	16	11.3	+.008	2.47	127	11.0	+.015	4.09	250
100	14	0	18.5	.96	60	50.0	+.001	2.71	150	58.5	−.006	3.01	156
		200	8.3	1.30	74	24.0	+.002	3.62	250	35.8	−.002	4.32	275
		100	8.3	.87	63	17.3	+.006	4.22	266	27.8	+.002	4.50	330
	50	0	13.4	1.47	76	36.6	−.003	2.93	220	45.4	−.005	3.71	302
		200	9.7	1.85	88	20.1	−.001	3.48	321	26.3	−.005	4.48	429
		100	9.6	1.75	95	16.3	+.003	4.06	274	19.8	+.001	4.37	399
	95	0	9.9	1.88	66	16.8	+.007	4.53	269	21.0	+.001	5.34	344
		200	6.8	1.75	94	11.7	+.005	4.53	321	13.6	+.003	5.37	421
		100	7.3	1.84	80	11.9	+.003	4.35	293	13.1	+.003	5.34	403
	in water*	0	7.2	.50	20	6.9	+.009	3.46	177	5.9	+.009	4.63	294
120	10	0	22.0	1.59	90	52.1	−.002	3.14	163	62.3	−.007	3.44	176
		200	8.9	1.90	102	28.4	−.001	3.85	205	40.4	−.006	4.74	292
		100	8.4	1.58	100	23.4	+.006	3.88	260	32.1	+.004	4.98	316
	50	0	17.1	2.37	136	40.6	−.004	3.99	227	47.4	−.008	4.18	247
		200	8.5	2.77	156	22.9	−.006	4.51	315	30.3	−.010	4.96	347
		100	8.0	3.05	130	18.1	−.004	4.38	335	23.7	−.007	5.03	400
	95	0	10.3	2.68	138	18.4	−.001	4.60	246	23.9	−.003	5.28	310
		200	7.1	2.50	150	14.1	+.001	4.63	269	17.9	+.001	5.33	332
		100	6.6	2.53	131	12.5	+.003	4.62	283	15.1	+.002	5.43	347
	in water*	0	9.8	0.34	24	9.2	−.001	4.93	209	9.1	+.005	5.99	362

*All water-cured specimens covered with wet burlap at room temperature during the first 8 hours.

LABORATORY STUDIES

Experimental laboratory work was confined to a study of the physical properties and behavior of air-entraining concrete during the first 7 days only, with particular attention to changes occurring during the first 24 hours after mixing. The major portion of the work consisted of two studies: the first to determine the effect of change in moisture condition at several different temperature levels on volume and the rate of development of strength and elastic modulus; the second to compare directly the membrane method with a standard wet method of curing with respect to warping, or curling, of concrete slabs under radiant heat.

Effect of storage conditions on physical properties of concrete

Scope of the tests—By storing concrete specimens at three different temperatures in air maintained at three different humidities and in water, a series of moisture contents and corresponding values of strength, elastic modulus and length were obtained at successive ages from 8 hours to 7 days. Temperatures of 75, 100 and 120 F were used to study the behavior of concrete in the summer temperature range. At each of these temperatures a separate group of specimens was stored in water and in air maintained as nearly as possible at relative

STORAGE DATA—3 x 3 x 15-in. BEAMS

2 days				3 days				7 days			
Moist. loss, percent	Length change, percent	Dyn. mod., 10^6psi	Flex. str., psi	Moist. loss, percent	Length change, percent	Dyn. mod., 10^6psi	Flex. str., psi	Moist. loss, percent	Length change, percent	Dyn. mod., 10^6psi	Flex. str., psi
58.0	−.013	4.28	191	60.5	−.014	4.42	185	65.2	−.024	4.25	248
24.7	−.004	5.50	393	27.1	−.005	6.51	421	34.5	−.010	7.33	430
20.3	+.006	6.04	423	22.7	+.008	6.44	569	27.9	−.001	7.47	512
49.1	−.003	4.15	239	51.2	−.006	4.26	224	55.0	−.013	4.63	299
27.4	+.003	5.38	337	29.9	+.002	5.62	418	37.0	−.006	5.84	557
17.5	+.003	5.43	465	19.6	−.001	6.13	503	24.7	−.003	6.32	568
22.3	+.001	6.03	407	24.2	.000	6.82	395	28.9	.000	7.40	420
16.1	+.004	6.20	442	17.2	+.005	6.80	456	20.2	+.004	7.33	552
14.1	+.003	6.10	406	14.8	+.005	6.77	495	18.3	+.002	7.18	638
9.3	+.011	5.81	427	8.5	+.013	6.24	532	6.7	+.014	7.52	757
63.8	−.014	3.65	187	66.5	−.015	3.81	218	71.6	−.023	3.92	243
43.5	−.007	5.15	323	49.0	−.011	5.13	320	57.0	−.023	5.12	373
34.8	−.002	5.58	419	38.7	−.005	5.48	401	49.3	−.018	5.60	412
51.6	−.009	4.28	310	54.1	−.006	4.24	310	58.5	−.016	4.27	351
35.0	−.008	5.41	459	37.8	−.010	5.58	433	45.9	−.018	5.47	456
25.8	.000	5.35	536	28.2	−.001	5.73	478	36.2	−.009	5.65	551
24.4	+.001	6.28	377	26.6	−.001	6.62	429	31.5	−.005	6.77	467
16.4	+.003	6.49	475	18.3	+.001	6.82	486	22.9	−.002	7.08	579
14.6	+.002	6.43	526	16.0	+.002	7.14	557	19.5	−.005	7.26	572
5.1	+.012	6.08	467	3.6	+.010	6.81	462	0.0	+.007	7.53	581
69.0	−.012	3.83	230	68.3	−.017	3.68	259	71.7	−.023	3.67	280
49.6	−.012	4.98	293	56.4	−.019	4.94	328	63.7	−.025	4.83	438
41.0	−.001	5.62	332	46.5	−.008	5.66	360	52.8	−.013	5.79	430
54.0	−.012	4.23	267	56.7	−.015	4.24	277	62.7	−.022	4.30	302
39.1	−.014	5.07	349	43.3	−.017	5.01	352	53.2	−.021	4.85	390
31.0	−.011	5.37	360	34.9	−.014	5.34	378	44.7	−.021	5.24	381
29.3	−.004	5.75	381	32.1	−.005	5.90	355	34.4	−.010	6.12	459
21.8	−.002	6.24	386	24.6	−.006	6.43	444	28.7	−.013	6.82	493
18.2	−.002	6.09	461	19.7	−.001	6.40	487	23.1	−.005	6.78	551
8.9	+.005	6.99	439	8.5	+.003	7.07	509	7.0	+.006	7.53	633

humidities of 10, 50 and 100 percent. The only significant deviation was at 75 F where the minimum humidity attainable was 30 percent.

Still further differentiation of moisture content was obtained by coating two groups of each series of specimens with a commercial membrane curing compound at coverage rates of 200 and 100 sq ft per gal. respectively. Details of materials, mix proportions and the procedure followed in making, storing and testing the specimens have been described elsewhere.[1]

Discussion—Quantitative data on the behavior of air-entraining concrete under a variety of environmental conditions make possible an approximate evaluation of different curing methods on the basis of the particular set of conditions produced by each. Also the several groups of specimens can be considered to represent elements of a concrete slab at different depths below the surface on a summer day and data from these tests will be used later for stress calculations.

The cabinet storage data are presented in Tables 1 and 2. Moisture losses at 7 days in most cases exceeded 20 percent of the original mixing water due to high initial losses before applying the membrane and to the high surface-mass ratio of the specimens. In most instances curing under water from 8 hours to 7 days was not sufficient to restore all of the moisture lost during setting and hardening prior to immersion.

TABLE 2—SUMMARY OF CABINET DATA—3 x 6-in. CYLINDERS

Curing conditions			8 hours		16 hours		24 hours		2 days		3 days		7 days	
Temp., °F	Rel. hum., percent	Seal, sq ft per gal.	Comp. str., psi	Moist. loss, percent	Comp. str., psi	Moist. loss, percent	Comp. str., psi	Moist. loss, percent	Comp. str., psi	Moist. loss, percent	Comp. str., psi	Moist. loss, percent	Comp. str., psi	Moist. loss, percent
75	30	0	39	15.8	174	52.5	438	63.2	548	66.8	626	68.6	903	71.7
		200	34	14.1	325	19.8	1170	24.8	1329	31.4	1805	33.5	2005	41.4
		100	37	14.7	428	17.5	965	20.5	1715	23.6	1805	25.9	2845	33.1
	50	0	78	18.7	326	48.8	580	57.0	749	60.6	803	61.1	994	63.6
		200	110	13.6	470	22.3	867	29.2	1266	34.4	1347	36.2	1778	41.1
		100	110	13.2	481	20.3	941	23.4	1485	27.2	1601	29.9	1920	34.3
	95	0	82	16.1	448	28.5	915	34.7	1119	37.2	1257	39.1	1636	42.3
		200	97	14.4	501	21.7	1092	24.2	1802	25.5	1845	26.7	2005	30.0
		100	90	14.2	597	18.7	1112	20.2	1552	20.3	2000	21.1	2175	23.7
	In water*	0	21	13.5	367	14.5	892	14.3	1330	13.6	1767	13.3	2130	10.9
100	14	0	261	21.4	467	60.7	651	65.9	641	72.3	705	73.4	726	77.1
		200	264	15.1	530	37.5	900	50.3	1145	57.8	1116	62.5	1154	70.8
		100	248	14.1	720	17.4	974	43.6	1287	51.2	1197	57.7	1463	68.3
	50	0	375	18.8	596	43.2	686	51.3	868	57.6	838	60.3	885	64.7
		200	561	15.1	725	27.7	994	35.3	1178	44.5	1170	47.1	1362	55.5
		100	467	15.1	835	23.5	1188	26.9	1169	32.8	1518	37.0	1873	45.3
	95	0	291	17.0	752	28.6	995	34.8	1165	38.0	1315	40.9	1553	44.7
		200	338	12.8	1040	20.7	1136	22.3	1554	25.3	1586	27.0	1827	30.6
		100	329	12.4	870	18.1	1345	19.2	1698	20.7	1823	21.7	2150	25.6
	In water*	0	29	12.1	517	11.9	863	11.4	1575	10.7	1797	9.9	2114	6.6
120	10	0	391	20.8	615	57.2	702	64.3	810	71.1	836	75.6	897	77.0
		200	494	13.4	728	47.3	1055	56.4	1057	64.5	1114	70.7	1134	75.6
		100	506	14.2	1006	36.9	1227	45.4	1302	50.8	1335	62.8	1459	70.6
	50	0	597	19.9	869	43.2	859	48.9	977	54.8	1053	57.3	1183	63.8
		200	830	12.4	973	28.6	1239	36.0	1508	41.5	1633	45.7	1793	54.8
		100	789	12.7	1128	20.8	1428	26.6	1760	31.8	1947	35.8	1913	44.3
	95	0	544	17.4	806	35.1	980	37.2	1174	41.4	1240	44.6	1305	49.2
		200	793	11.5	1055	23.3	1585	27.1	1780	32.1	1860	34.1	2105	39.5
		100	657	11.6	1104	21.6	1330	22.9	1730	25.4	2070	28.0	2100	32.8
	In water*	0	31	14.8	852	15.7	1200	14.7	1467	14.8	2011	14.7	1915	14.9

*All water-cured specimens covered with wet burlap at room temperature during the first 8 hours.

Even at the relatively large moisture losses observed, the strength and modulus of elasticity of the concrete were not adversely affected to any considerable extent until moisture losses of 20 to 25 percent were exceeded.

The data in Table 1 also show generally that there was no shrinkage from the 8-hour length at any test age or temperature when moisture losses remained below about 20 percent; conversely, moisture losses in excess of 20 percent usually, but not always, resulted in a net contraction. Strength and dynamic modulus increased rapidly during the first day, especially the first 16 hours, and were accelerated by higher temperatures. At the lower humidities maximum flexural strength and dynamic modulus were attained quickly at a low level due to rapid drying but little increase took place after 3 days; at the higher temperatures there was little gain after 36 hours. Rapid drying is sharply reflected in the prematurely stunted development of strength and elasticity, indicating virtual cessation of hydration at an early age.

Specimens stored in water and in air at 95 percent relative humidity exhibited a more regular gain and better concordance, and reached higher 7-day strengths than the others, better concordance probably being due to a more uniform distribution of moisture within the specimens at all ages. At 120 F more moisture was lost at the same relative humidities than at 75 F and, in general, early strengths were higher and 7-day strengths were lower.

Warping tests

Scope—The principal object of these experiments was to measure the warping of 8 x 12 x 84-in. concrete beams cured by clear membrane, white membrane and wet burlap when exposed to radiant heat on the top surface only. Four series of three beams each were cast: Series 1, 2 and 3 to compare the three curing materials mentioned above on waterproofed, dry sand and saturated sand subbases respectively; and Series 4 on a saturated sand subbase to compare clear membrane, no curing, and dry burlap over clear membrane, dry burlap being added in the latter case solely for heat insulation.

A bank of incandescent bulbs provided sufficient radiant heat to produce a maximum temperature of 120 to 130 F in the top surfaces of beams cured with clear membrane. Details of materials and procedures used in these tests may also be found in the publication already referred to.[1]

After curing, the beams were broken in the center, removed from the subgrade and the two halves tested in flexure using a centerpoint loading on a 30-in. span. Immediately after the flexural tests, the fractured surface was sprayed with phenolphthalein in absolute alcohol to determine the depth of desiccation. Data for these tests are given in Table 3 and photographs of the beams of Series 1 are shown in Fig. 1, in which the desiccated layer appears as a thin white line across the top of the section. This line is not nearly as well defined in the beam cured with white membrane which indicates less surface desiccation in this beam than in the other two of the same series.

Each beam end was subjected to a wear test using the apparatus illustrated in Fig. 2. A load of 18 lb was applied to the wearing tool by suspending a 1-lb weight from the end of the drill press arm. Data are given in Table 4, each value representing the average of 18 tests.

Warping—The time-warping curves of Fig. 3 through 6 show that the beams kept continuously moist with wet burlap varied least in temperature and warped least of any in the four series of tests. Beams cured with clear membrane are at the opposite extreme, while those with white membrane

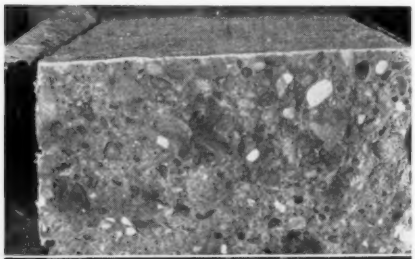

Fig. 1—Surface desiccation of beams on waterproofed subgrade. Desiccated layer shows as a thin white line across the top of the fracture. Upper—Clear membrane, 10 days. Middle—White membrane, 10 days. Lower—Wet burlap 7 days, air 3 days

coatings show a substantial reduction of both temperature and warping from these upper extremes. It should be pointed out that the intention in these experiments was to compare the two types of membrane curing with an ideal method rather than with burlap curing as such. Wet curing of concrete pavements for 7 days is extremely unusual in modern practice, 3 days being the ordinary limit, and a more realistic comparison would further take into account the continuity of water supply to the concrete, an important factor in the problem.

All beams exhibited a permanent upward curl almost from the moment of final set. At no time after the first 10 hours, except for a few periods of maximum surface temperature in the burlap-cured beam of Series 1, was any beam warped concave downward. This can be attributed to the influence of (1) temperature gradients at the time the concrete is passing from the plastic to the elastic state, (2)

TABLE 3—FLEXURAL STRENGTH AND SURFACE DESICCATION OF BEAMS USED IN WARPING TESTS

Series	Subgrade	Duration of curing, days	Curing treatment	Flexural strength, psi	Thickness of desiccated layer, in.
1	Waterproofed	10	Clear membrane	511	1/8
			White membrane	528	1/16
			Wet burlap*	690	1/16
2	Dry sand	10	Clear membrane	578	1/8
			White membrane	596	1/16
			Wet burlap*	687	1/16
3	Saturated sand	7	Clear membrane	563	1/16
			White membrane	655	Slight
			Wet burlap	725	Slight
4	Saturated sand	7	Clear membrane	634	1/16
			Dry burlap on clear	588	1/16
			No curing	536	1/16

*Wet burlap removed at the end of 7 days.

moisture gradients and (3) differential autogenous volume change.

Vertical temperature differences of 15 degrees or more were established in the 8-in. slabs before final set took place and produced a differential thermal volume change which occurred initially without appreciable stress because of the plasticity of the concrete in this interval. The data in Table 1 show that concrete at 100 F had already attained approximately one-fourth of its 7-day modulus of elasticity at 8 hours and at 120 F the ratio was still greater. Thus, an elastic structure is being established when a considerable temperature difference exists between the top and bottom of the slab, and the concrete is becoming rigid with the top layer in an expanded condition. Any later reduction of the temperature gradient tends to produce uplifting of the beam ends.

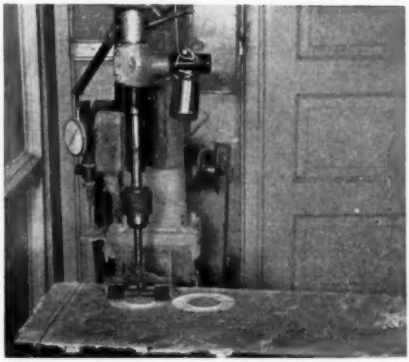

Fig. 2—Beam end being subjected to wear test

Moisture gradients effective in producing warping are not established until after the constant-rate period of evaporation is over, which corresponds roughly to the final setting time of the cement. When the plane of evaporation falls below the surface, however, a differential drying shrinkage takes place which tends to counteract temperature effects during daylight hours, but which adds to the forces causing upward movement of the slab ends after the heat source is removed. The time at which these various phenomena occur is important and it should be remembered that *while, in general, no significant moisture gradient exists up to the time when final set takes place, a temperature gradient very near the maximum has been built up in the interim.* Little real compensation for temperature gradients, therefore, is brought about by moisture gradients during the first day, since the concrete has already set in a heat-distorted structure before moisture effects come into play.

Evaporation of moisture does exert a direct compensatory influence on the warping of the slab, however, through its cooling effect on the surface. This is well illustrated by the warping curves of Fig. 6, which show consider-

TABLE 4—SUMMARY OF WEAR TESTS

Curing method	Penetration, in.			
	1 min.	3 min.	5 min.	10 min.
Clear membrane	0.032	0.067	0.092	0.138
White membrane	0.026	0.060	0.075	0.123
Wet burlap	0.025	0.048	0.071	0.117

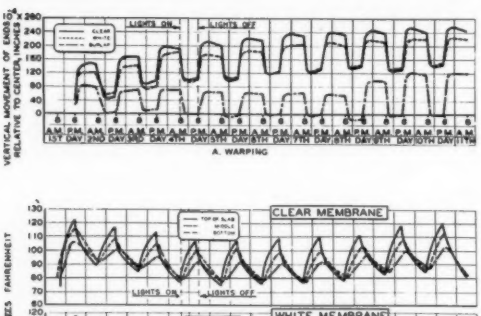

Fig. 3—Temperature gradients and warping in beams of Series 1—Waterproofed subgrade

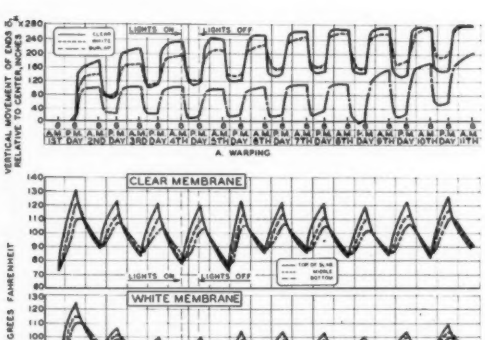

Fig. 4—Temperature gradients and warping in beams of Series 2—Dry Sand subgrade

ably lower temperatures during the first day for the air-cured beam than for the membrane-cured one. The same graphs also demonstrate that the warping pattern is fairly well established during the first few hours of exposure to radiant heat when it is observed that the end deflections of the air-cured beam remained below those of the membrane-cured one during the entire 7 days in spite of the fact that both maximum temperatures and temperature gradients attained in the former equaled after the first day and subsequently exceeded those in the latter.

The third factor to be considered in the interpretation of the warping data is autogenous shrinkage, or the contraction in absolute volume of the cement-water system accompanying the hydration reactions. Differential autogenous shrinkage may arise from (1) heterogeneity of the concrete and (2) different rates of hydration through the depth of the slab induced by temperature and moisture gradients. The effect of the latter is probably more or less transitory, although it may be significant in the early behavior of the pavement, but that of the former is permanent. Although the

use of air-entraining concrete has resulted in much better control of segregation and bleeding, the surface layer of finished concrete still is likely to differ materially from the bottom, especially in cement content, porosity, and volume response to changes in temperature and moisture. Some investigators have observed that a residual upward curl remains in concrete slabs dried from both surfaces, even when no temperature gradient exists.[2,3] Apparently this differential volume change does not progress far in concrete kept continuously moist after finishing. Once allowed to dry, however, the slab cannot be restored to its initial flat condition by addition of water to the top surface.

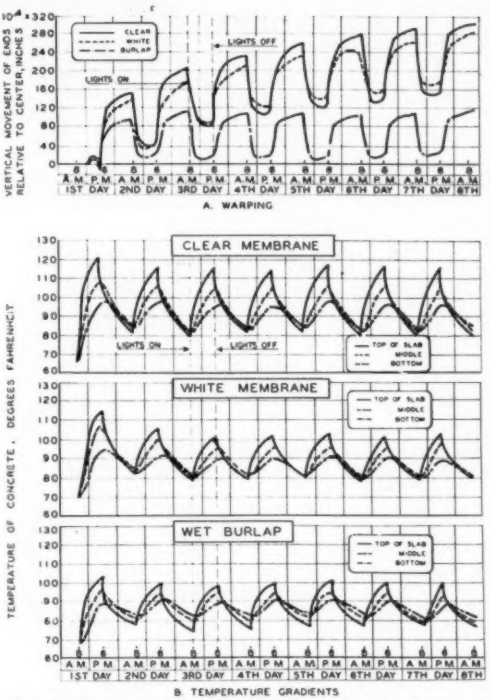

Fig. 5—Temperature gradients and warping in beams of Series 3—Saturated sand subgrade

The total warping at any instant is the resultant of the effects produced by temperature gradient, moisture gradient and differential autogenous volume change. When the heat source is removed, all three factors contribute to upward movement of the slab ends.

The curves of Fig. 3 through 6 also emphasize the significance of the time of taking initial or base measurements. Obviously results may vary widely, depending upon the particular time chosen for reference measurements. In these experiments, the first readings were taken 5 to 6 hours after finishing and curling may have already begun. From the data shown, it seems unlikely that slabs poured on bright summer mornings would ever actually be warped concave downward after the concrete had hardened except under the most unusual temperature and moisture conditions.

The effect of the removal of the wet burlap is shown in the curves for Series 1 and 2. As soon as the covering was removed, the temperature differentials and warping amplitudes increased greatly. The burlap-cured beam of Series 2 rapidly assumed a progressively increasing residual set but this effect was much subdued in the corresponding beam of Series 1.

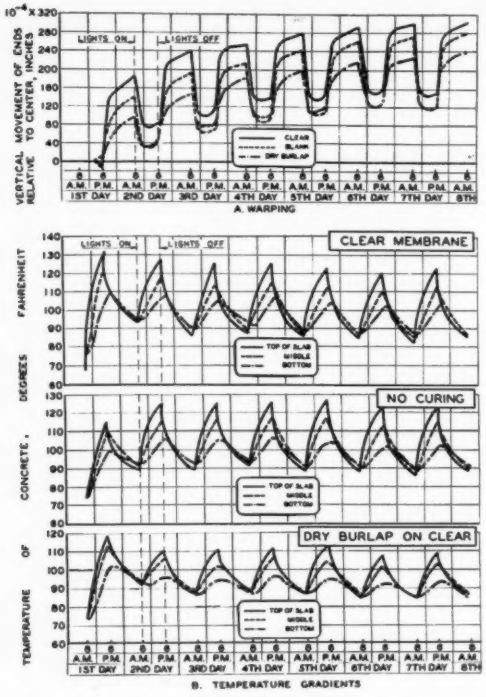
VERTICAL MOVEMENT OF ENDS RELATIVE TO CENTER, INCHES
LIGHTS ON
LIGHTS OFF
CLEAR
SLANK
DRY BURLAP
A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M. P.M. A.M.
1ST DAY 2ND DAY 3RD DAY 4TH DAY 5TH DAY 6TH DAY 7TH DAY 8TH
A. WARPING
TEMPERATURE OF CONCRETE, DEGREES FAHRENHEIT
CLEAR MEMBRANE
NO CURING
DRY BURLAP ON CLEAR
TOP OF SLAB
MIDDLE
BOTTOM
B. TEMPERATURE GRADIENTS

Fig. 6—Temperature gradients and warping in beams of Series 4—Saturated sand subgrade

Temperature control—The use of white-pigmented membrane reduced maximum temperatures about 15 degrees below those attained in beams cured with clear compounds. The reduction in temperature differential is reflected in correspondingly lower deflections of the slab ends. It should be kept in mind that lower temperatures reduce moisture losses by lowering the vapor pressure of the evaporable water in the concrete, and lower moisture losses are partially responsible for the decrease in warping.

To reduce temperatures during the curing period, a dry burlap covering was applied as soon as possible to the surface of a beam (Series 4) that had been previously sprayed with clear membrane curing compound. A temperature reduction of 15 degrees followed and resulted in a 26 percent decrease in warping when compared with beams cured with clear compound only. White-pigmented compound gave comparable results.

Strength and abrasion resistance—The data of Table 3 indicate that satisfactory flexural strengths were obtained except in the two membrane-cured beams of Series 1. There is no apparent explanation for the anomaly that lower strengths were developed in beams cast on a waterproofed subgrade than in those cast on a dry subgrade and cured by identical procedures. As was to be expected, the strengths of wet-cured beams were consistently higher than the others, and the beneficial effect of the saturated subgrade on strength is also noticeable.

Results of the wear tests presented in Table 4 show that resistance to surface abrasion was affected somewhat by the curing methods employed. The average depth of penetration of the wearing tool into the burlap-cured concrete was 17.5 percent less than in concrete cured with clear membrane and 5 percent less than in white membrane-cured beams. For concrete roads,

resistance to wear is a secondary consideration, since the pavement will ordinarily succumb to traffic and weather long before it is worn out.

FIELD STUDIES

Studies of field curing were of three types differing in purpose and method of approach. The first was intended to provide temperature data; the second was concerned with the practicability of the use of white-pigmented curing compounds on pavements; and the third consisted of the acquisition and interpretation of data on the condition of recently constructed concrete pavements cured with clear membrane compounds.

Temperature study

On September 9, 1947, temperature measurements were made on five pavement slabs placed at intervals between 7:30 a.m. and 3:30 p.m. inclusive. This two-lane pavement was 22 ft wide, 9 in. thick and constructed according to current Michigan State Highway Department design which, in the present instance, provided for 100-ft contraction joints with load transfer dowels, no expansion joints, and 86 lb of steel reinforcement per 100 sq ft.

Surface temperatures at all five stations are charted in Fig. 7, and both top and bottom temperatures at the first station appear in Fig. 8. The short vertical lines drawn through the curves indicate the time of application of the clear membrane.

Considering first the graph of top and bottom temperatures at the single station, Fig. 8, the following facts may be noted:

1. Between 12 noon and 1:00 p.m. a vertical temperature difference of about 20 F had already been established, and this difference was maintained until the top surface began to cool at 3:00 p.m.

2. Final set of the cement probably occurred somewhere between 12 and 1 o'clock, about 4½ to 5½ hours from the time of mixing, the setting reactions being accelerated somewhat by the increase in temperature.

3. The temperature difference decreased rapidly after the maximum temperature in the top surface was reached at 3:00 p.m., then vanished at about 6:00 p.m. after which a reversal occurred and the difference increased in the opposite direction gradually to 7 F at 2:00 a.m.

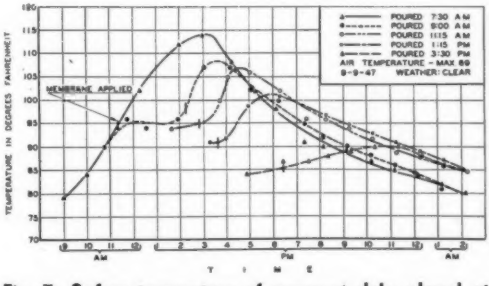

Fig. 7—Surface temperatures of pavement slabs placed at different times of the day.

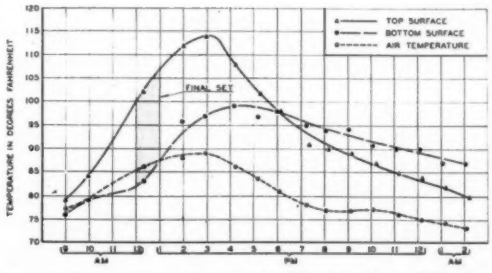

Fig. 8—Temperatures of pavement slab placed at 7:30 a.m.

4. At 12 midnight, the temperature in the bottom of the slab was approximately the same as at the time of final set and was slowly decreasing.

Bearing the foregoing facts in mind, an attempt may be made to determine the conditions existing in the first slab of the day's pour at midnight of the same day.

In this particular pavement the concrete was fully restrained until the 100-ft, weakened-plane contraction joints opened by cracking. Therefore, the tensile stress in the slab before the ends were freed is given by

$$S_t = Eet \quad \ldots\ldots (1)$$

in which S_t is the unit stress due to temperature, E and e are the modulus of elasticity and thermal coefficient of expansion of the concrete respectively, and t the change in average temperature of the slab.

The thermal coefficient of expansion of concrete containing natural aggregates is close to 5×10^{-6}, and the dynamic modulus and flexural strength may be taken from Table 1 as 4.5×10^6 and 300 psi respectively. Now the resistance of the concrete to direct tensile forces is not more than half its flexural strength, or about 150 psi at this stage. Substitution in Eq. (1) shows that a drop of only 7 degrees in the average temperature of the slab would be sufficient to crack the joints. From the temperature curves of Fig. 8 it seems certain that the joints should have cracked by midnight.

Next, the temperature differential effective in producing warping at any instant must be calculated under the condition that no warping stress due to temperature exists when the temperature differential prevailing in the concrete at the time of final set is present. Mathematically, the effective temperature differential may be expressed thus:

$$t_w = T_t - T_b - (T_{ht} - T_{hb}) \quad \ldots\ldots (2)$$

where t_w = temperature differential effective in causing warping,
T_t, T_b = temperatures in the top and bottom surfaces respectively at the given time and
T_{ht}, T_{hb} = temperature in the top and bottom surfaces respectively at the time hardening takes place.

A reasonable approximation of the maximum warping stress due to vertical temperature difference in a slab of sufficient weight to prevent it from warping away from the subgrade except near the ends may be found from

$$S_w = \frac{Eet}{2} \quad \ldots\ldots (3)$$

where S_w = maximum warping stress,
E = modulus of elasticity of the concrete,
e = thermal coefficient of expansion of the concrete and
t = difference in temperature between the top and bottom surfaces of the slab.[6]

At midnight, then, $t_w = -5 - 20 = -25$ F, E and e are taken as 4.5×10^6 psi and 5×10^{-6} respectively and the warping stress due to temperature only is found from Eq. (3) to be 281 psi. Since the warping stress is directly proportional to the difference in temperature between the top and bottom of the slab, in the present case a reduction of 12 degrees in the difference would decrease the warping stress approximately 50 percent.

Finally, the subgrade resistance to slab movement after the ends are freed remains to be calculated. Theoretically, the forces of frictional restraint exerted when the pavement is cooling cause maximum tensile stress at the midsection of the slab. The amount of tension thus created at the center may be calculated roughly by equating the frictional force developed for unit width over half the slab length to the tension existing in the concrete as follows:

$$\frac{Wh}{12} \times \frac{L}{2} \times f = 12h \times S_f$$

$$\text{or} \quad S_f = \frac{WLf}{288} \qquad (4)$$

where S_f = tensile stress in psi,
W = unit weight of concrete, lb per cu ft,
L = length of the slab in feet,
f = coefficient of subgrade resistance and
h = thickness of the slab in inches.

Assuming a friction coefficient, f, of 1.0, and the unit weight of concrete to be 150 lb per cu ft, the maximum frictional stress at the midsection is found from Eq. (4) to be 52 psi. This determination presupposes that a sufficient drop in temperature exists at the bottom of the slab to cause appreciable contraction at the subgrade surface and that dowel friction at the joints can be neglected.

Taking the maximum tension from subgrade resistance as 52 psi, the net warping stress due to temperature amounts to 281 minus 52, or 229 psi. This figure does not take into account the two other factors contributing to warping in the same direction, *i.e.*, moisture gradients and differential autogenous volume changes. Since the flexural strength of the concrete at the age considered is about 300 psi, the induced stresses in this example are dangerously near the rupturing point.

These conditions are by no means the most severe possible in hot weather concreting. The sun was far past the summer solstice and substantially higher temperatures and temperature gradients could be expected earlier in the season, especially in late spring when the ground is still relatively cool and the sun is hot. The foregoing is only a rough approximation but points out the magnitude of the stresses that may be expected in summer concreting under a definite set of conditions and shows the need of adequate temperature and moisture control to minimize early cracking or incipient cracking of the pavement.

The temperature curves of the five slabs placed at different times of the same day (Fig. 7) require little comment. It may be observed, however, that the application of the membrane to the slab placed at 9:00 a.m. was unduly delayed with the result that the temperature did not rise above 96 F until after 2:00 p.m. and then reached a maximum of about 108 F between 3:00 and 4:00 p.m. Nevertheless, this slab may have fared worse than the first one because of excessive surface drying. The two slabs placed in the

afternoon were in a much better position to come through the first 24 hours in a structurally sound condition than were the three placed during the morning, other things being equal.

The Grand Ledge curing experiment

The study of white membranes was made on the Grand Ledge Experimental Project as a part of a broader investigation planned primarily to evaluate new standards of pavement design. For use on pavements, the possible effects of the white pigment on operator vision and the change in general appearance due to weathering and traffic are important and the principal object of this study was to learn something of these effects.

Scope—Two consecutive 100-ft slabs in the south lane of the 22-ft 8-in. pavement were cured with a white compound and the corresponding slabs in the north lane with the clear compound furnished on the job. Both white and clear membranes were applied at the rate of 200 sq ft per gal.

Thermocouples were placed in the concrete at the top, middle and bottom of the slab in each lane and seven inserts 40 in. apart for clinometer warping measurements were set along the outside edge of both lanes of the pavement for 20 ft on each side of the common contraction joint. Initial readings were taken approximately 6 hours after the finishing operation, and at various intervals for 3 weeks thereafter. The pavement was also inspected several times to note the general appearance and weathering of the white membrane.

Results and discussion—The curves in Fig. 9 were plotted from clinometer readings to represent the longitudinal profile of the end portions of the two slabs for both white and clear membranes with reference to the point (No. 7) farthest from the joint. After 32 hours, the minor irregularities disappeared and there was only upward warping at the joint edge. Different temperature gradients changed the magnitude but not the direction of warping, which is in agreement with the laboratory studies. The concrete was lifted at progressively greater distances from the ends as drying continued and its flexural strength increased until at 3 days the curvature extended more than 20 ft back from the joint, as indicated by the increased slope at point 7.

There was little difference in the warping of the slabs cured with white and clear membranes. It was late in the season and the highest temperatures measured in the tops of the slabs were only 97 F for the clear and 87 F for the white membrane, with a 10-degree differential in each case. Probably the dowels and tie-bars were equalizing influences also.

The white membrane weathered away rapidly and uniformly without mottling or other unsightly weathering effects. Inspection also revealed that at no time after opening of the pavement to traffic was there any objectionable glare from the white surface.

The results of the experiment strongly indicate that there should be no serious problems encountered in the use of white compounds, and that their use would materially enhance the structural stability of concrete pavements cured with membranes.

Survey of postwar pavements

Condition surveys have been made of 29 concrete pavements built during 1946 and 1947, all of which were cured with clear membranes. The results

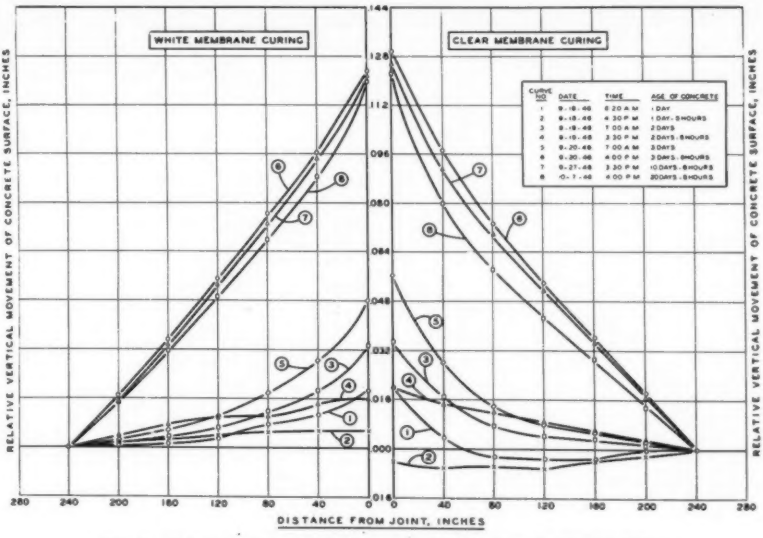
WHITE MEMBRANE CURING
CLEAR MEMBRANE CURING
RELATIVE VERTICAL MOVEMENT OF CONCRETE SURFACE, INCHES
DISTANCE FROM JOINT, INCHES
CURVE NO. | DATE | TIME | AGE OF CONCRETE
1 | 8-18-46 | 6 20 A M | 1 DAY
2 | 8-18-46 | 4 30 P M | 1 DAY - 8 HOURS
3 | 8-18-46 | 7 00 A M | 2 DAYS
4 | 8-19-46 | 3 30 P M | 2 DAYS - 8 HOURS
5 | 8-20-46 | 7 00 A M | 3 DAYS
6 | 8-20-46 | 4 00 P M | 3 DAYS - 8 HOURS
7 | 8-27-46 | 3 30 P M | 10 DAYS - 8 HOURS
8 | 10-7-46 | 4 00 P M | 200 DAYS - 8 HOURS

Fig. 9—Early warping of 100-ft pavement slabs at a contraction joint

of these surveys have been given in detail in the basic report[1] and show that in all of the pavements built during the spring and summer months, early cracking, when occurring at all, was found predominantly in the morning pours. Pavements constructed in the fall did not, in general, exhibit this tendency.

For all of these projects, representing a wide range of materials, weather conditions, location and constructional details, there were approximately three times as many cracked slabs in the morning pours as in the afternoon pours. This means that, potentially at least, the total cracking at early ages could have been reduced about 50 percent by effective temperature control.

COMPARISON OF CURING METHODS

An excellent summary of the advantages and disadvantages of membrane curing based on considerable experience and research has been given by Blanks, Meissner and Tuthill[4] and will not be repeated here. The results of the present investigation, however, permit a more quantitative evaluation of some of the factors enumerated by them, particularly with respect to stresses induced by differential heating. In the following comparison of membrane and wet curing methods, the relative merits of the two will be judged on the basis of three general qualities, namely, efficiency, practicability and cost.

Efficiency

Efficiency as used here is the extent to which a particular curing method fulfills all of the requirements set forth in the definition and function of curing when conditions are controlled so as to secure optimum results.

Wet curing is unquestionably more efficient than membrane curing for hot weather concreting. The temperature-regulating effect of water is great and, together with the continuous external supply of moisture to maintain a maximum rate of hydration and prevent undue shrinkage, will insure minimum induced stress and maximum strength gain during the early hardening period. From the viewpoint of moisture requirements, curing by membrane is entirely adequate but some means of early temperature control during hot, clear weather should be provided. Probably the most practical way of doing this is by the use of white-pigmented compounds. Other satisfactory methods are 24-hour initial moist curing, the use of heat-insulating coverings of various kinds, and shifting construction operations to a later period of the day. All practical means of reducing temperatures and temperature gradients should be employed in hot weather concreting, regardless of the curing method used.[5]

For cold weather construction, membrane curing has advantages over wet methods. There is less freezable water in the cement paste and the concrete is therefore better able to withstand lower temperatures without frost damage. Furthermore, the restriction of evaporation by the membrane appreciably preserves warmth in the slab and, in conjunction with some insulator like dry straw or earth, will provide considerable protection against freezing. At such times, additional water is neither necessary nor desirable.

Practicability

By practicability is meant not only feasibility but also what may be termed control expectancy in actual practice. In addition to its usual meaning, it denotes the extent to which the field operation may be expected to provide the conditions necessary for optimum results from the method.

The preceding definition applied to wet curing raises a number of implications including (1) the difficulty of supplying curing water for a mile or more of pavement behind the mixer; (2) laxity in keeping the covering wet during the curing period; (3) difficulty in maintaining inspection sufficiently vigilant and forceful to see that the covering is kept wet; and (4) resistance shown by contractors, to whom wet curing is a troublesome and expensive operation. In the laboratory warping tests described earlier, it was necessary to saturate the burlap at least once every hour to keep it moist during the day. It must be admitted that such practice is seldom, if ever, found in general concrete pavement construction.

Membrane curing is not without practical problems also, but they present much less difficulty than those of wet curing. The major problem in the early days of membrane curing was uniform distribution of the compound on the pavement surface. Traveling distributors with shielded spray nozzles have largely overcome this trouble and the equipment is being progressively

improved. Rate of coverage must be watched also and it is wise to check the amount of curing material used against the surface area covered. A third factor requiring close attention is the time of application of the material. Curing equipment should operate well up behind the finishers, and the compound should be sprayed on as soon as the surface moisture has disappeared. Undue delay results in excessive surface drying which may combine with temperature effects to crack the pavement. It results also in discontinuous films caused by the compound penetrating the concrete surface rather than bridging the voids. When delay is unavoidable, the pavement surface should be resaturated with water before applying the membrane.

Cost

It is difficult to obtain comparable data on costs of clear membrane and wet curing methods, principally because the two types have not been contemporaneous. Membrane curing with clear compounds did not come into wide use until the beginning of the war and its adoption as a temporary alternate was accompanied by an almost complete cessation of the use of other methods in Michigan. From such data as are available from contractors, however, it is safe to say that the cost of membrane curing is not more than one-half that of wet curing. At the present time, the cost of wet curing is almost prohibitive when added to the already inflated costs of concrete pavement construction, and this situation is probably aggravated by the unwillingness of contractors to return to the older more cumbersome methods.

SUMMARY

The more important findings of this investigation are given below. Some are derived from statements in the original report[1] but are included here for convenience.

1. The original water content of a plastic concrete paving mix is approximately double the amount required for hydration of the cement compounds. Water in excess of minimum hydration requirements must be available, however, as a medium for continued hydrolysis and hydration reactions during curing.

2. Retention of about 80 percent of the original net mixing water at the end of a 7-day curing period is sufficient for continued hydration and development of adequate strengths during the period.

3. In most regions of this country, the method of artificial curing used does not significantly affect the ultimate strength or behavior of pavement concrete at advanced ages.

4. The method of curing does affect the physical properties and structural stability of pavement concrete at early ages, however, through its influence on temperature and moisture gradients within the slab. On hot summer days the effects of temperature during the early hardening period are critical, but the occurrence of temperature cracking in a pavement of given design depends not only on the rate and extent of temperature change but also on the character of the subgrade, the physical and chemical properties of cement and aggregates, and the proportioning, mixing, placing and finishing of the concrete.

5. There is considerable evidence to show that temperature cracking is apt to occur more frequently in sections of pavement placed in the morning than in the afternoon on hot summer days. When using membrane curing compounds, such cracking can be appreciably reduced

by (1) the use of white-pigmented curing compounds; (2) initial 24-hour curing with wet burlap, followed by saturation with water and application of membrane; (3) the use of dry, protective coverings of other types in conjunction with membrane curing compounds, including shading; or (4) by shifting construction operations to a later period of the day.

6. Pavements placed in hot weather exhibit concave upward warping only, the degree of which is determined largely by the magnitude of the temperature gradient existing when the concrete sets and hardens, and the time of application of curing materials.

7. Although curing with wet coverings under ideal circumstances still provides the most favorable conditions within the concrete during the setting and early hardening period, ideal conditions are extremely difficult to realize in practice and are seldom obtained on the job. For this reason, more uniformly satisfactory results are to be expected from the use of membrane curing compounds than from methods which demand constant attention and supervision during the entire curing period.

8. For fall or early winter construction in temperate climates, membrane curing preserves warmth in the slab through restriction of evaporation, and creates a condition of minimum saturation at the threshold of the freezing season.

9. There was no significant difference in surface hardness or wear resistance of concrete beams cured with wet burlap and white membrane. Beams cured with clear membrane exhibited somewhat lower resistance than the others.

10. Available data show that the unit cost of membrane curing before the war was about half that of wet curing methods.

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Is mixing equipment adequate for special mixes now used? Tests on tilting mixers indicate possible improvements.

Blade Changes Improve Tilting Mixer*

By GLENWAY MAXON†

SYNOPSIS

Recent experiments, as well as earlier studies, on changing the blading of tilting concrete mixers so as to improve the quality of the mixed concrete are described. The evolution of the blade shapes and the effect of these changes on the path of the materials through the mixer are illustrated.

INTRODUCTION

Those connected with the concrete industry have added immensely to the knowledge of concrete in the past ten years through investigation of the chemistry of cement, water-cement ratio, gradation of aggregates, and through durability and other studies. But the knowledge of mix design acquired in the laboratory is of little value unless it is carried out in the mixer and in the equipment used for charging the mixer and placing the concrete.

As air-entraining agents and pozzolanic materials are added to a mix, there is an accompanying reduction of cement, water and sand, and mixing becomes increasingly difficult. This is because the blading designed to intermingle streams of comparatively plastic ingredients *is not capable of intermingling harsh aggregates as easily* but must pick up the aggregates, particularly the heavy ones, and drop them to the bottom in the plane of rotation from which they were picked up. In other words, dry and harsh aggregates do not move easily from one end of the mixer to the other.

The writer believes that the tilting mixer offers the best chance of improving mixing action. Primarily because more free space is available for mixing action; and, secondly, because it is not necessary to pick up the aggregates, carry them to the top of the mixer and drop them. This mixer is built for either front or rear end charging. The axis of the mixer is horizontal when used as a rear end charger. The mixer is tilted slightly up when used as a front end charger. Generally speaking, only 31 percent of its volume is occupied by the shrunken mixture.

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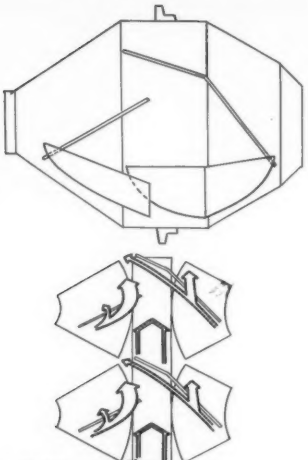

Fig. 1—Early blading arrangement of tilting mixer. Note lack of free motion toward the charging end.

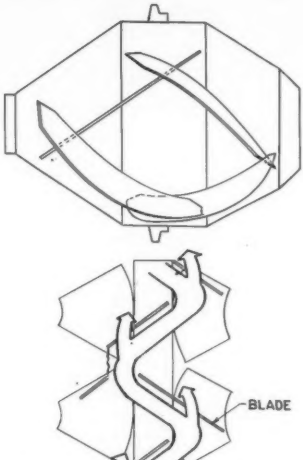

Fig. 2—Alternate blade positions in tilting mixer. This arrangement permitted segregation of water and heavy aggregates.

The tilting mixer has been built for many years with herringbone-positioned blades which restrain the concrete and carry it up as the mixer turns, allowing the ingredients to cascade from an intermediate position before the blades are at their uppermost point. It is believed that much can be done to improve the action of this mixer. In the first place, the action of various blading can be photographed, and in the case of the rear end charger can be studied from both ends. Past experience has been that the blades on the discharge end tended to screw large stone back into the mixer while the mixer was discharging. This characteristic led to making the discharge blades smaller which resulted in longer required mixing time, particularly if water preceded aggregates by too long an interval. Under these conditions, the portion of the aggregates in the discharge end remained wetter than the concrete in the rest of the mixer for as long as two minutes.

IMPROVEMENTS IN MIXING ACTION

The first blade changes aimed at improved efficiency in this type of mixer

were made at Parker Dam. Wayne Byrne, an engineer with the Bureau of Reclamation, played a part in the changes there; and he suggested some of the changes made in 1943 on a 2-cu yd tilting mixer at Altus Dam. Although others suggested that grinding of large aggregates could be lessened by blade changes, these were the first steps in the attempt to improve mixing of concrete in this type of tilting mixer.

In January 1949 the Corps of Engineers and the contractor permitted changes of blading in one of the two mixers at Allatoona Dam. Slow motion pictures showed improved mixing but indicated further possible improvement. Later, M. C. Kolinski of Milwaukee and the writer built a special 3-cu yd tilting mixer designed, among other purposes, to permit the maximum study of mixing action. This mixer was placed in operation in August 1949.

The accompanying diagrams illustrate the flow of concrete in old type mixers and the changes made to improve mixing action.* While the type of mixer illustrated in Fig. 1 had free motion from the charging end to the discharging end, it lacked free motion in the opposite direction. A long time was taken in drifting the aggregates near the discharge end back toward the loading end.

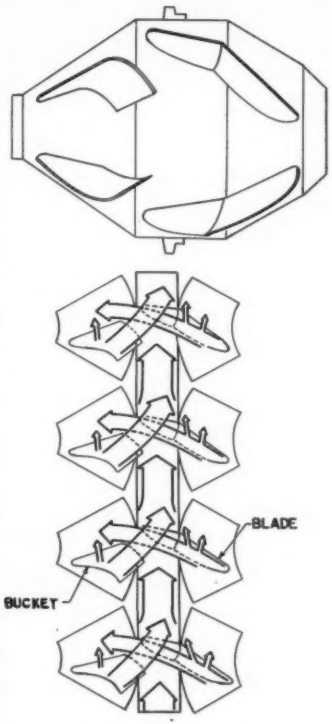

Fig. 3—Blade arrangement to allow interchange and stream diversion. Folding action provides better mixing.

Fig. 2 shows the action with alternate blade positions. The resulting action might be compared to that of a wave which moves up and down; the concrete moving towards the discharge end and then moving towards the charge end, oscillating more than mixing. The water tended to float near the front if too much preceded the aggregates. It appeared that to prevent the oscillating wave motion it was necessary to fold some of the ingredients from near the discharging end of the mixer over the ingredients from the charging end while they were being forced toward the discharge end.

Fig. 3 shows the changes made to replace oscillation with interchange and stream division. The slope of the cone was increased on the discharge

*In the figures, the double cone and cylinder represent the drum in plan or elevation with some of the blades omitted. Segments of the charge and discharge cones are unwrapped and laid beside the cylindrical part, mapping the entire blading. Superimposed on these maps is the path taken by the ingredients after they have been mixed to a state slippery enough to move on the blades as well as spill. The comparative width of the arrows shows relative proportions of stream divisions and the direction taken.

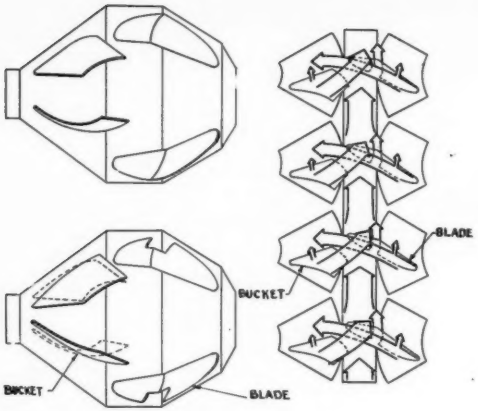

Fig. 4—Addition of bucket-shaped discharge blade and change in curvature results in effective shrink mixing

end from 30° to 38°. This permitted replacing the typical blade with a bucket-shaped blade designed to lift the aggregates, cement and water and spill them over the charging-end blade just as the latter passes its portion of the concrete toward the discharge end of the mixer. Fig. 4 indicates the additions made to the bucket-shaped discharge blade and the change in curvature. This mixer shrink mixes all the materials in less than 20 seconds from the start of loading.

From these initial studies others have been suggested, such as investigation of the need for more than one type of blading. Highway work calls for a dry concrete with relatively small aggregate. Concrete to be pumped must be of a wetter consistency. It is reasonable to believe that each should have special mixer blading. We know that this is true for the dry, harsh mix containing 6-in. stone required for dam construction.

REQUIREMENTS FOR TILTING MIXERS

Criteria for mixing concrete in the tilting mixer include:

1. All ingredients should have an equal chance to get to all parts of the mixer.
2. Velocity of flow should be as uniform as possible with many diversions and crossings of paths.
3. Some paths and diversions must travel to both extremities and, therefore, the extremities should not be too remote from one another.
4. Dropping from near the top of the revolution causes grinding and possibly the splashing tends toward segregation.
5. Brackets holding the blades to the mixer should be streamlined, otherwise they cause a dead spot in the mixing action and the building up of solid concrete.
6. A mixer should not require special charging procedures.
7. Various mixes require different blading. Harsh mixes with large aggregates require less blading and probably should require longer mixing.

Before construction of the Walnut Lane Bridge, a full-size beam was tested to destruction. The test results verified the safety of the design.

Prototype Prestressed Beam Justifies Walnut Lane Bridge Design*

By GUSTAVE MAGNEL†

SYNOPSIS

The tests made on a prestressed concrete beam of 154 ft 8 in. span, identical to the beams of the main span of the Walnut Lane Bridge in Philadelphia, more than justified the adoption of prestressed concrete for the bridge. The test methods and results are described. The test beam exhibited a safety factor against cracking of about 2, which would be far lower for a reinforced concrete beam. The factor of safety against complete failure was about the same as for reinforced concrete while the deflection was less. It is possible to use prestressed concrete for beams where structural steel can not be used due to excessive deflection. The tests proved that the Walnut Lane Bridge will have an exceptional degree of safety with less weight and greater durability than would be possible with reinforced concrete, as well as being considerably cheaper than the conventional solution.

INTRODUCTION

The first prestressed concrete bridge in the United States is now under construction at Philadelphia. It is of the Blaton-Magnel type with cables anchored at their ends by sandwich plates. The main span is 154 ft 8 in. center to center of bearing plates and is composed of 13 prestressed beams placed alongside each other and prestressed transversely so as to make them all deflect together as a solid slab.

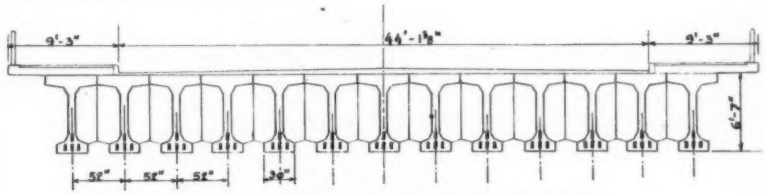

Fig. 1—Cross section at midspan of main span of bridge

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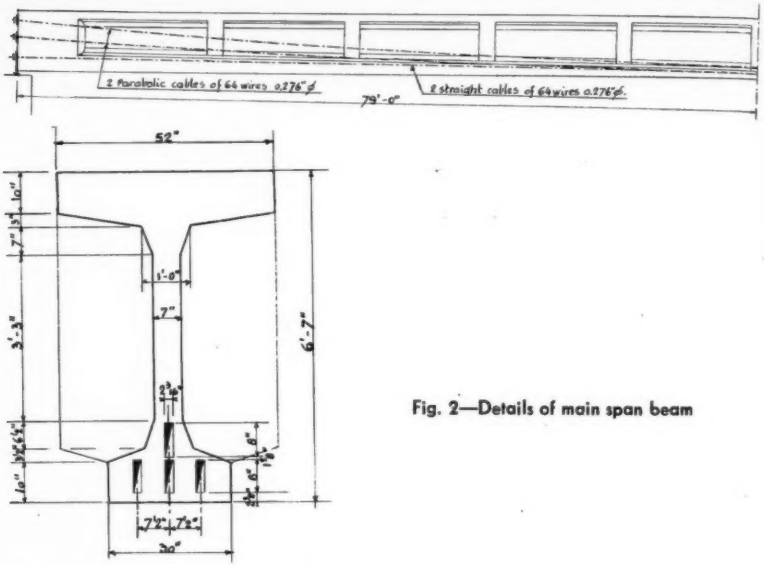

Fig. 2—Details of main span beam

Fig. 1 shows a cross section of this main span and Fig. 2 the shape and dimensions of one of its girders.

The specifications required that the contractor, before being allowed to go ahead with the actual work, make a beam identical to those to be used in the bridge, but solely for testing purposes under direction of the author.

DESCRIPTION OF TEST GIRDER

Fig. 2 shows that the girder is neither prismatical nor straight. Its cross section is generally I-shaped with the lower flange narrower than the top. The web is only 7 in. thick. The girder is prestressed by 4 cables, each of 64 wires of 0.276 in. diameter, stressed initially up to 131,000 psi. The web has a light reinforcing mesh of ordinary mild steel. The top flange also has a light transverse reinforcement of mild steel.

The beam is provided on each side with eight stiffeners through which—in the actual bridge—will be threaded the cables for the transverse prestressing. On each end are what are generally called end blocks, against which are placed the distributing plates situated under the sandwich plates. The beam was supported on one end on a fixed steel bearing plate, and on the other end on a lubrite plate allowing expansion with little friction.

MATERIALS

The beam was cast in a timber form. Ready-mixed concrete, with a slump of about 2 in., was compacted by both internal and external vibration.

TABLE 1—PROPERTIES OF CONCRETE USED IN TEST BEAM

Curing conditions	Compressive strength, psi	Modulus of elasticity, psi
A—16 days moist cured, 1 day dry	5160 5470 5470 5660	3,500,000
B—20 days moist cured, 1 day dry	5550 5540 5320 5650	3,700,000
C—38 days curing in the open on the job	5350 5200	4,400,000* 3,600,000†

*At zero stress.
†At 3600 psi.

The crushing resistance and modulus of elasticity of the concrete, measured on standard cylinders is given in Table 1. These data are plotted in relation to strain measurements in Fig. 3 to 5.

The author would like to point out that the figures found for the modulus of elasticity are abnormally low when compared with his own experience and also when compared with those deduced from the slope deflection and strain measurements during the test.

The wire was cold drawn steel, having the following characteristics: diameter—0.276 in.; equivalent yield stress—220,000 psi; tensile strength—247,000 psi; and modulus of elasticity at 126,000 psi—27,000,000 psi. The stress-strain diagram is given in Fig. 6.

GENERAL DESCRIPTION OF TEST METHOD

The beam was loaded by eight jacks distributed along it, as shown in Fig. 7. These jacks acted upward against transverse steel girders attached by vertical rods to dead weights placed on the ground in eight places along each side of the girder.

The eight loads given by these jacks were always kept identical and increased by computed and controlled increments. These loads were measured up to 1400 lb per ft of beam by strain gages placed vertically on small steel

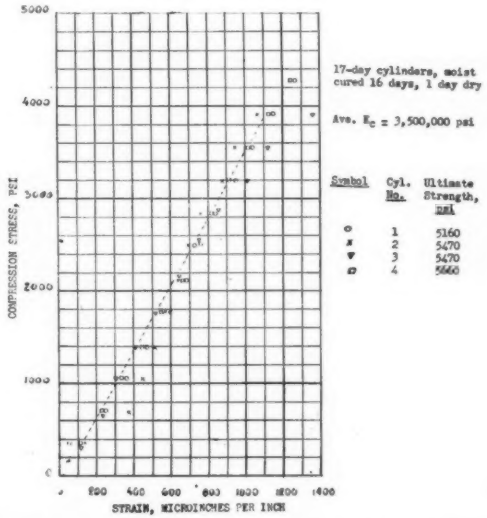

Fig. 3—Compression tests for E_c, 17-day cylinders—Strains measured by Ames dial gages

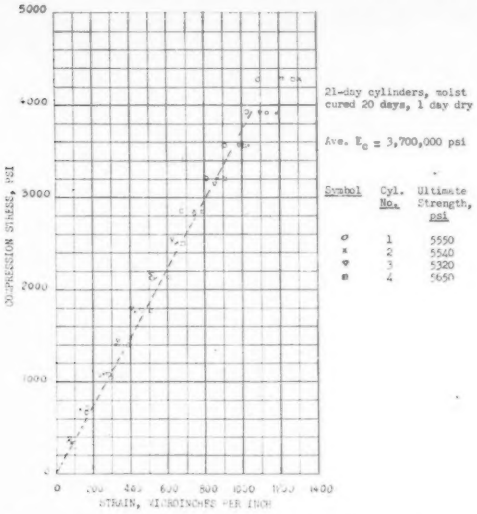
21-day cylinders, moist cured 20 days, 1 day dry
Ave. E_c = 3,700,000 psi
Symbol | Cyl. No. | Ultimate Strength, psi
o 1 5550
x 2 5540
▽ 3 5320
□ 4 5650
COMPRESSION STRESS, PSI
STRAIN, MICROINCHES PER INCH

Fig. 4—Compression tests for E_c, 21-day cylinders—Strains measured by Ames dial gages

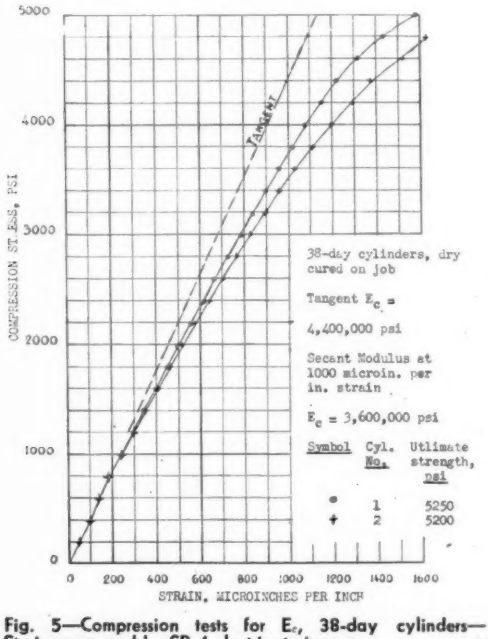
Tangent
38-day cylinders, dry cured on job
Tangent E_c = 4,400,000 psi
Secant Modulus at 1000 microin. per in. strain
E_c = 3,600,000 psi
Symbol | Cyl. No. | Ultimate strength, psi
• 1 5250
+ 2 5200
COMPRESSION ST.ESS, PSI
STRAIN, MICROINCHES PER INCH

Fig. 5—Compression tests for E_c, 38-day cylinders—Strains measured by SR-4 electric strain gages

cylinders inserted between the jacks and the steel girder. For higher loads, the readings on the manometers of the jacks were considered accurate enough, the possible error being about 10 percent.

The writer regrets that it was not possible to test the beam by using 8 jacks worked by one single pump. He has, in Belgium, worked with up to 26 jacks of 200 tons each, placed on the same pump in the delicate operation of raising a tower 8 ft without demolishing it.

Twenty - three strain gages were placed on the surface of the concrete as shown in Fig. 8.

During the prestressing operation, the deflections of the beam were measured by three fleximeters placed at midspan, at quarter point and one 8 ft from the opposite end of the span. During the load test of the beam, the vertical deflections were measured at quarter point, midspan and at the other quarter point. The slopes at the ends of the beam were measured by two clinometers.

When the beam carried a load of 4500 lb per ft, it was not yet broken and it was impossible to immediately add further loads, as the dead weight

placed on the ground was not heavy enough. Two days later the beam was loaded by a concentrated load at midspan in addition to the action of six of the eight jacks until the beam broke.

During the load test, the beam carried, besides its own weight, (a) the weight of the jacks and the steel girders underneath and (b) the load given by the jacks. It should be noted that the jacks were pumped up to the point where each gave, as the total of (a) + (b), a load of 5200 lb. These eight loads produced at midspan a bending moment identical to the one given by a distributed load of 250 lb per ft. This is called the equivalent distributed load (equivalent inasmuch as it gives the same bending moment as the real eight point loads). When this equivalent load of 250 lb per ft was reached all measuring apparatus for slopes, strains and deflections were set to zero.

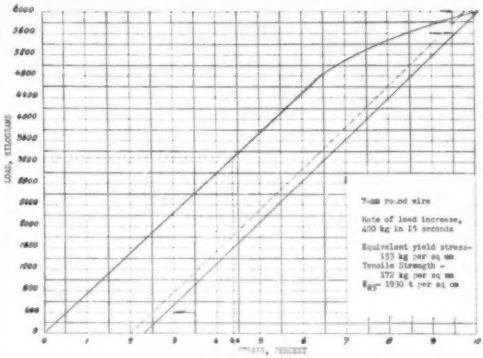
7-mm round wire
Rate of load increase, 400 kg in 15 seconds
Equivalent yield stress- 153 kg per sq mm
Tensile Strength - 172 kg per sq mm
E_{av} 1930 t per sq cm

Fig. 6—Stress-strain diagram of prestressing wire

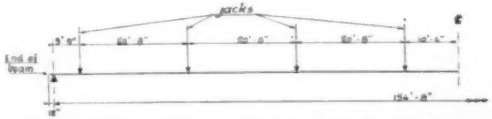

Fig. 7—Arrangement of jacks for test loading of beam

The dead weight of the test beam gave bending moments at midspan of 5,360,000 ft-lb and at the quarter points 4,060,000 ft-lb. The midspan moment corresponds to an equivalent load of 1780 lb per ft of which the grout accounts for 60 lb per ft.

The beam was 31 days old when the test started and 36 days old when it was broken. The prestressing took 6 days and was started when the concrete was 15 days old.

CALCULATED STRESSES

At midspan due to prestressing alone

The steel area is 15.3 sq in.; the initial steel stress, 131,000 psi; hence the prestressing force is 2,000,000 lb.

The eccentricity of the cables is 38.45 in.; the concrete area, 1349 sq in., and the moment of inertia, 1,086,000 in.^{4}; the centroid of the concrete section is 31.8 in. below the top fibers.

Hence the stresses in top fibers are -770 psi and $+4820$ psi in the bottom fibers. In time these stresses decrease owing to the shrinkage of the concrete and the creep of steel and concrete.

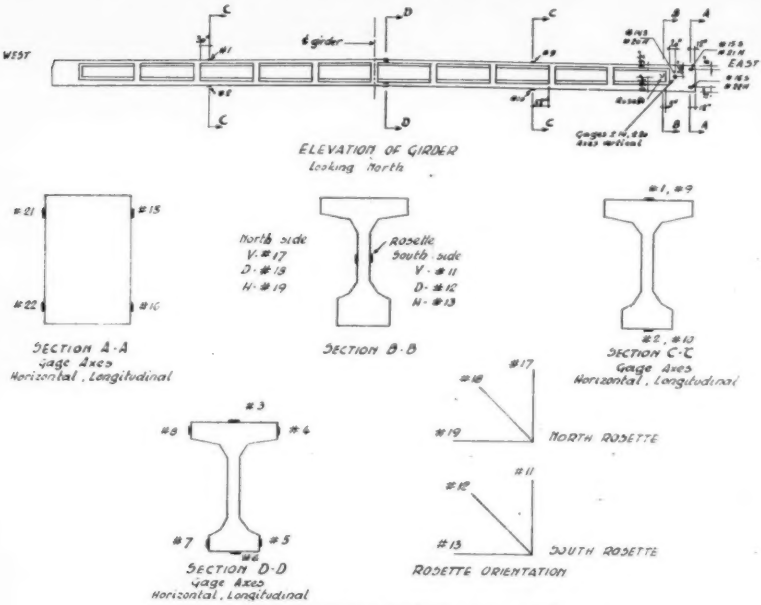

Fig. 8—Arrangement of strain gages on test beam

At midspan due to dead weight existing at the moment of prestressing

This load gives a bending moment of 5,150,000 ft-lb. and hence stresses (no grout at that moment) of +1815 psi in the top fibers and −2690 psi in the bottom fibers.

At midspan due to loads applied after prestressing

The dead weight of the total bridge deck must first be considered, which means for each beam a load of 364 lb per ft in addition to the one considered in the previous paragraph, the total giving a moment of 1,088,000 ft-lb including the moment due to the weight of the grout. There is further the superimposed load equivalent to 446 lb per ft giving a moment equal to 1,330,000 ft-lb.

In computing the stresses due to these two moments, a larger moment of inertia must be considered than previously, since, when the corresponding forces act on the beam, the grouting in of the cables has already been done and the beam works, in fact, as does a reinforced concrete beam, with a factor $n = 5$. This gives a moment of inertia of 1,201,000 in.^{4} and a depth of the centroid of the cross section equal to 33.9 in.

Hence we can compute the following stresses. For a moment of 1,088,000 ft-lb the stress in the top fibers is +369 psi, while in the bottom fibers it is −492 psi. For a moment of 1,330,000 ft-lb the stress in the top fibers is +451 psi, and that in the bottom fibers is −602 psi.

Combined stresses

From the above the following combined stresses are found.

A. Initially

Beam under dead load

Top fibers	− 770	+ 1815	=	+1045 psi
Bottom fibers	+4820	− 2690	=	+2130 psi

Bridge under dead load

Top fibers	+1045	+ 369	=	+1414 psi
Bottom fibers	+2130	− 492	=	+1638 psi

Bridge fully loaded

Top fibers	+1414	+ 451	=	+1865 psi
Bottom fibers	+1638	− 602	=	+1036 psi

B. In the long run with 13 percent loss of prestress

Bridge under dead load

$+1414 + (0.13 \times 770) = +1514$ psi
$1638 - (0.13 \times 4820) = +1011$ psi

Bridge fully loaded

$1865 + (0.13 \times 770) = +1965$ psi
$1036 - (0.13 \times 4820) = +409$ psi

In test beam at the start of the load test

It has been said previously that the moment at midspan was 5,360,000 ft-lb due to dead load alone. This moment gives stresses of +1890 psi in the top fibers and −2800 psi in the bottom fibers. Combined with the prestressing stresses, this gives initially $-770 + 1890 = +1120$ psi in the top fibers and $4820 - 2800 = +2020$ psi in the bottom fibers. Note that at the beginning of the load test a certain loss of prestress must already have occurred. Its value will be found by considering the cracking load of the beam.

ACTION OF THE EIGHT JACK LOADS

The eight loads P distributed as shown in Fig. 7 give bending moments (in ft-lb) at midspan of 144 P and 108 P at quarter points if P is expressed in lb. The distributed load p (in lb per ft) that gives the same bending moment as the eight loads P is given by $p = P/20{,}764$.

So, for example, if $P = 5{,}191{,}000$ lb, $p = 250$ lb per ft. The moments are then 747,600 ft-lb at midspan and 560,600 ft-lb at quarter points. The maximum shearing force is 20,764 lb.

CRACKING LOAD OF THE BEAM

Before prestressing started, the beam showed a crack about 17 ft from midspan. This was obviously due to some settlement in the timber supporting it. Consequently, in calculating the cracking load, the tensile stress of the concrete was not taken into account and the load was simply calculated giving a zero stress in the bottom fibers at 17 ft from midspan. This computation gives the prestressing stress of +4800 psi, load at prestressing, −2550 psi and at 250 lb per ft, −339 psi.

If η is the part of the prestress remaining at the moment of the test ($\eta < 1$) and if one knows that the first crack occurs at an equivalent load of 1400 lb per ft, then

$$\eta \times 4800 - 2550 - 339 \frac{1400}{250} = 0$$

hence $\eta = 0.92$.

It may be admitted that during the test the loss of prestress was 8 percent which compares to the ultimate loss of 13 percent allowed for in the design.

Note that the cracking load, or rather the load when the existing crack opened, is accurately known since a strain gage (No. 23) was placed across the crack and this allowed an accurate determination of the load at the moment when the crack opened.

SAFETY OF THE BRIDGE AGAINST CRACKING

The dead load of the bridge corresponds to an equivalent load for each beam of 1780 − 60 + 364 = 2086 lb per ft. The superimposed load corresponds to 446 lb per ft.

The first crack occurred at an equivalent load of 1400 lb per ft which was applied simultaneously with the dead weight of the test beam of 1780 lb per ft. Hence at cracking the beam carried a superimposed load (over and above the dead load as in the bridge) of 1400 + 1780 − 2086 = 1094 lb per ft. This corresponds to a safety factor against cracking of 1094/446 = 2.65 if the dead load is neglected.

If the total load was taken into account, the safety factor would be

$$\frac{1094 + 2086}{446 + 2086} = 1.25$$

This is for the test conditions, *i.e.*, at the moment when a loss of prestress of 8 percent has occurred.

These safety factors will decrease and it is interesting to know what they will be ultimately. Of course, this can be found by computation, as follows. Ultimately the stress in the bottom fibers at the section 17 ft from midspan will be 0.87 × 4800 = 4170 psi. The load at prestressing gives a stress of −2550 psi, 250 lb per foot gives a stress of −339 psi. Hence the equivalent load p which will give a zero stress is given by

$$4170 - 2550 - 339 \frac{p}{250} = 0$$

Hence, p = 1195 lb per ft.

This means a total load on the beam of 1780 + 1195 = 2975 lb per ft or a superimposed load in the bridge of 2975 − 2086 = 889 lb per ft. This gives a factor of safety of 889/446 = 2.0 based solely on the superimposed load. If the dead weight is taken into account, it becomes

$$\frac{889 + 2086}{446 + 2086} = 1.18$$

WORKING LOAD OF THE BEAM

In the actual bridge one beam has to carry a dead load of 2086 lb per ft and a superimposed load of 446 lb per ft or a total of 2532 lb per ft. During the test, the beam weighed 1780 lb per ft. Hence, the working load conditions are reached when the eight isolated forces P of Fig. 7 reach a value corresponding to an equivalent load of $2532 - 1780 = 752$ lb per ft, say 750 lb per ft.

The double working load conditions are reached when the equivalent load corresponding to the eight forces P is equal to 1192 lb per ft, say 1200 lb per ft.

PRINCIPAL TEST RESULTS

In the following paragraphs attention will be paid, not to all test results, but to the principal ones, and mainly those corresponding to working load conditions, double working load conditions and breaking conditions. In this way, an over-all picture of the results will be given. After that, more complete data will be given for the measurements made, but without discussion and only for the information of those who might wish to study further the detailed test results.

Calculated deflections and slopes of beam under load

Consider the beam loaded with eight point loads P as in Fig. 7. In the computation of the deflections at midspan and at quarter points as well as of the end slopes, the fact that the beam is not prismatical has been taken into account. The following values were found under an equivalent load of 250 lb per ft.

$$\text{Deflection at midspan} = \frac{2{,}580{,}000}{E} \text{ in. } (E \text{ in psi})$$

$$\text{Deflection at quarter span} = \frac{1{,}830{,}000}{E} \text{ in.}$$

$$\text{Slope at ends} = \frac{4380}{E} \text{ in.}$$

Values of E computed from measured deflections and slopes

During the first loading of the beam, the following values were found for an increment of equivalent load ranging from 250 to 750 lb per ft, this latter giving the working load conditions.

Deflection at midspan = 0.695 in.
Deflection at quarter points = 0.471 in.

These correspond with the calculated values found above for $E = 7{,}420{,}000$ psi for midspan and $E = 7{,}770{,}000$ psi for quarter points.

$$\text{Slope at ends} = \frac{4.0 \times 2 \times 3.14}{60 \times 360} = 0.001166 \text{ in.}$$

This corresponds with the calculated value found above for $E = 7{,}530{,}000$ psi. These three values of E are nearly the same so that it seems reasonable to take their average, say $E = 7{,}570{,}000$ psi.

The reader will compare this value with the one mentioned above as result of tests on cylinders (about 3,600,000 psi). The value based on the deflection measurements seems to the writer more reliable as it corresponds to what he has always found on concrete of the quality used in this case.

The same calculation is now applied to the findings during the second loading of the test beam, namely, for increments of equivalent load ranging from 250 to 1200 lb per ft, the latter corresponding to double working load conditions. This gives similarly from deflection at midspan, E = 6,380,000 psi and from slope at ends, E = 6,750,000 psi, say an average of E = 6,530,000 psi.

This average value is smaller than the one found for the load range of from 250 to 750. This should be so, as it is known that E decreases when the stresses increase.

Values of *E* computed from strain gage measurements

First, let an equivalent load be considered varying from 250 to 750 lb per ft. This gives moments at midspan of 1,495,000 ft-lb and at quarter points of 1,121,000 ft-lb. The corresponding stresses at midspan are +506 psi in the top fibers and −678 psi in the bottom fibers while at quarter points they are 371 psi for top fibers and 467 psi for bottom fibers.

The strain gages gave the following results at midspan: top fibers—80 micro-in. per in. (No. 3) and bottom fibers—110 micro-in. per in. (No. 6); at quarter points: top fibers—67.5 micro-in. per in. (No. 1-9) and bottom fibers—77.5 micro-in. per in. (No. 2-10).

The corresponding values of E are at midspan: top fibers—6,310,000 psi and bottom fibers—6,160,000 psi; at quarter points: top fibers—5,500,000 psi and bottom fibers—6,020,000 psi. Now let an equivalent load varying from 250 to 1200 lb per ft be considered. One finds similarly at midspan: top fibers—6,150,000 psi and bottom fibers—6,430,000 psi; at quarter points: top fibers—5,880,000 psi and bottom fibers—6,250,000 psi.

All these values are of the same relative magnitude and much closer to those deduced from the deflection and slope measurements than those found directly from compression tests of cylinders.

Strain gage readings during prestressing

In the writer's opinion, little can be deduced from the readings made on any of the strain gages during prestressing. Indeed, the conditions of the support of the beam during this operation are unknown. Normally, when a beam is resting on its formwork and is then prestressed, it lifts itself up automatically and instead of bearing continuously on the forms, it gradually takes the position where it rests only on its natural end supports. These conditions were not established during the prestressing of the test beam because the end bearings had to be changed and during that time the beam was supported on timber cribbing reducing the span considerably. Consequently the stresses due to dead load could not develop completely, but no one knows to what extent they did develop.

Remember, for example, that in the bottom fibers at midspan, the prestressing alone developed a compressive stress of 4820 psi and that the dead load should have developed at that moment a tensile stress of 2690 psi the resulting stress being 2130 psi compression.

If we now assume that the dead load developed only half of 2690 psi, the resulting stress becomes 4820 − 1345 = 3475 psi, or 63 percent more than in the first case.

It is known that the strain gage recordings in Table 2 are the difference between the readings before prestressing and those after the girder was finally placed on its final end supports and had no intermediate supports anywhere. But this does not change the fact that during prestressing, stresses as high as 3500 psi must have been reached instead of the 2130 that should have been reached. This changes the whole strain development.

On the other hand, it should be remembered that strain gages are not suitable for measurements extending over a period of 6 days as were those during prestressing, owing to the plasticity of the material fixing the gage to the concrete.

This explains why the E, corresponding to the 560 micro-in. per in. measured at midspan on the lower fibers (gage 6), is only equal to 3,800,000 psi; far too low for the quality of concrete used.

What happened after cracking load

When the first crack occurred at an equivalent load of 1400 lb per ft, it was decided to go on loading until the crack was wider. In fact, the load was carried to only 1500 lb per ft when local cracks occurred in one of the panels next to the central stiffener. See Fig. 9 for details.

This secondary effect was probably due to one of the external cables not being straight in its passage through this panel, giving a slight outward force dislocating one side of the lower flange. This occurrence was considered as of no importance whatever for the future of the test, as at that moment the bottom flange was no longer of any necessity as it worked in tension. As a matter of fact, a bit of it might have fallen off without changing in any way the ultimate load of the beam. However, it was decided not to unload the beam any more and leave it at double working load of 1400 lb per ft. Actually, in taking off this load the bottom flange would have to resist compression again and the cracked concrete might have buckled outside and broken. In that case the beam unloaded would no longer have been symmetrical and would have taken a big horizontal deflection.

TABLE 2—STRAINS DURING PRESTRESSING*

Gage No.	Strain, micro-in. per in.	Gage No.	Strain, micro-in. per in.	Gage No.	Strain, micro-in. per in.
1	315 C	9	240 C	17	5 T
2	770 C	10	640 C	18	55 C
3	280 C	11	30 T	19	110 C
4	310 C	12	315 C	20	60 C
5	480 C	13	485 C	21	50 C
6	560 C	14	20 C	22	105 C
7	635 C	15	45 C	23	—
8	320 C	16	65 C		

*Prestressing operation lasted five days.

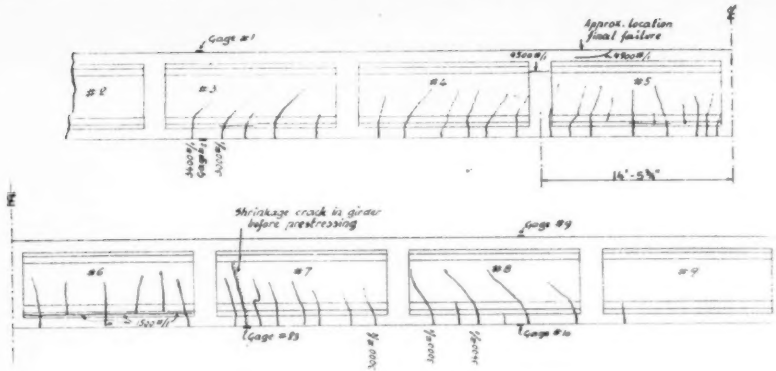

Fig. 9—Part elevation of beam showing crack pattern

Breaking load

Once the cracking load of 1400 lb per ft had been reached, the loads given by the eight jacks were no longer measured by strain gages fixed on steel cylinders placed between the jacks and the steel girders on which they were acting, but by the manometers of the jacks. Hence, these measurements were not as accurate as before. It is thought that these manometers indicated loads about 10 percent too high and this is taken into account in the following analysis.

On the first day, the equivalent load of 4500 lb per ft was reached and the beam did not break. Its deflection under this load was about 17 in. Two days later the beam was broken by placing on it loads corresponding to an equivalent load of 5000 lb per ft. Just before breaking, the deflection was about 27 in. and the beam broke then by crushing of the concrete. The corresponding bending moment was 14,940,000 + 5,360,000 = 19,300,000 ft-lb.

Factor of safety

Let it be remembered that, in the bridge, each beam carries a permanent load of 2086 lb per ft. Under test conditions the dead weight of the beam was 1780 lb per ft, hence the superimposed load capable of causing the breaking of a beam in the bridge is equal to 5000 + 1780 − 2086 = 4694 lb per ft. This corresponds to a safety factor of 4694/446 = 10.3 if one neglects the dead load. If, on the other hand, the total load is taken into account, the factor of safety is $\frac{4694 + 2086}{446 + 2086} = 2.69$. These figures correspond to a loss of prestress of 8 percent.

Actually, if the loss of prestress were 13 percent, they will be slightly different, probably a little higher. The stress in the steel will decrease while the crushing strength of the concrete will be higher.

Calculated breaking load

Several theories have been developed for finding the breaking moment of a reinforced concrete beam or a prestressed beam. None of these is perfect as all require the knowledge of a number of coefficients about which little is known.

As the beam broke by crushing of the concrete, it is safe to assume that at that moment the stress diagram on the concrete was rectangular with a stress of about 5500 psi.

The force acting in the steel wires can never exceed the ultimate strength or $15.3 \times 247{,}000 = 3{,}770{,}000$ lb. If this force did develop, the concrete near the top of the beam would be compressed by the same force, which means that the height of the compressive zone would be at least $\frac{3{,}770{,}000}{51 \times 5500}$ = 13.2 in. Consequently, the greatest possible value of the lever arm is $79 - 8.75 - 13.2/2 = 63.65$ in., giving a moment of resistance equal to $3{,}770{,}000 \times 63.65/12 = 20{,}000{,}000$ ft-lbs. This is the maximum moment that can be expected. The actual breaking moment, as found at the end of the test, was 19,300,000 ft-lb.

Let us now calculate the breaking moments in accordance with the "Ultimate Design of Reinforced Concrete" published by the Portland Cement Assn.

We know that $f'_s = 5500$ psi and $f_c = 247{,}000$ psi. Hence the plastic ratio is

$$\beta = \frac{1}{1 + \left(\frac{5500}{4000}\right)^2} = 0.346$$

The steel ratio is

$$p = \frac{A_s}{bd} = 0.00427$$

The neutral axis ratio is then

$$k = \frac{2\,p\,f_s}{(1+\beta)\,f'_c} = 0.280$$

and the moment arm ratio

$$j = 1 - \frac{1 + \beta + \beta^2}{3(1+\beta)}\,k = 0.898$$

Hence the ultimate moment is

$$M = A_s f_s jd = 19{,}850{,}000 \text{ ft-lb.}$$

which is near the maximum of 20,000,000 ft-lb calculated above.

Dynamic measurements of *E*

The modulus of elasticity was measured by putting the girder under vibration and taking an oscillogram of it; this gave 2.16 cycles per second (f_n). Using the formula

$$f_n = \frac{\pi}{2} \times \sqrt{\frac{EI}{\mu l^4}}$$

where μ is the mass per unit length, $E = 6{,}550{,}000$ psi.

The change in over-all length of the girder during prestressing was also measured and, based on this, E was found to be 3,100,000 psi. However, it has been explained why this is not reliable.

Strain gages on ends of beam

Not much can be deduced from the measurements made with the strain gages used on or near the end-blocks. There is no equality in the readings made at corresponding points on both vertical sides of the endblocks. For example, during prestressing, gage 11 read 30 and gage 17 read 5, gage 13 read 485 and gage 19 read 110, gage 14 read 20 and gage 20 read 60, and gage 16 read 65 and gage 22 read 105. Under 500 lb per ft of equivalent load between 25 and 750, gage 13 read 10 and gage 19 read 0, gage 14 read 10 and gage 20 read 15 and gage 16 read 10 and gage 22 read 5. Some readings changed over from compressive to tensile when the loads increased, for example, gages 20, 21 and 22.

The prestressing lasted 6 days and strain gage readings over such a long period are seldom reliable.

The only conclusion to be drawn from the test, as far as the endblocks are concerned, is that their factor of safety is higher than that of the beam at midspan; indeed when the beam broke there was no sign of either cracking or crushing in or near the endblocks.

CONCLUSIONS

From this test the following conclusions were reached.

1. Up to cracking load, the beam behaved fairly well in agreement with the calculations, much better than is the case with a reinforced concrete beam.

2. The calculated breaking load was close to that obtained in the test.

3. It is reasonable to assume, in computing the safety against cracking, that the beam is cracked before prestressing, and consequently that the concrete has no tensile resistance. Cracks before prestressing due either to shrinkage or settlement of the forms are almost unavoidable.

4. Notwithstanding the previous conclusion, the safety factor against cracking is ultimately about 2 if superimposed load is considered only, and 1.18 if total load is considered; whereas it would be far lower for a reinforced concrete beam.

5. The factor of safety against breaking is as high as 10.3 when one considers only the superimposed load and about 2.7 when one considers the total load. This would be about the same for reinforced concrete.

6. The deflection is less than it would be for a reinforced concrete beam.

7. It is possible to make a road bridge of 155-ft span with a ratio of span to depth of beam equal to 23.5 which appears to be impossible in structural steel, mainly owing to excessive deflection.

8. At an age of about 32 days the loss of prestress due to all causes is equal to 8 percent.

9. The Walnut Lane Bridge is a structure having an exceptional degree of safety; it is much lighter and its durability is far better than would be the case with ordinary reinforced concrete.

10. The maximum compressive strain measured just before breaking was 1.280 micro-in. per in., say 1.28 in 1000. The maximum tensile strain measured without cracking is 375 micro-in. per in., say 0.375 in 1000. These figures are worth noting as little data are available on this particular aspect.

11. Owing to the above conclusions and to the fact that the prestressed solution was more than 31 percent cheaper than the conventional solution, the City of Philadelphia was more than justified in adopting prestressed concrete which, although new in the United States, has proven its value for many years in Europe.

12. The American technicians were able to make a perfect job of this first prestressed concrete girder.

ORGANIZATION

The project is under the direction of A. Zane Hoffman, Chief Engineer of the Bureau of Engineering, Surveys and Zoning of the City of Philadelphia. The author is consulting engineer to the Preload Corporation, contractor for the prestressed concrete superstructure. All strain, deflection and slope measurements were made by Arthur R. Anderson, consulting engineer.

APPENDIX

TABLE 3—STRAINS IN MICRO-IN. PER IN. UP TO WORKING LOAD

Gage No.	Equivalent load, lb per ft			
	250	750	750, held 30 min.	250, unload
1	0	70 C	85 C	15 C
2	0	75 T	70 T	5 T
3	0	80 C	90 C	10 C
4	0	90 C	95 C	15 C
5	0	110 T	110 T	5 T
6	0	110 T	115 T	10 T
7	0	105 T	110 T	10 T
8	0	90 C	100 C	20 C
9	0	65 C	75 C	5 C
10	0	80 T	80 T	10 T
11	0	10 T	5 T	5 C
12	0	15 C	15 C	5 C
13	0	10 T	5 T	5 T
14	0	10 C	5 C	5 C
15	0	5 T	5 T	5 T
16	0	10 T	5 T	5 T
17	0	10 T	10 T	10 T
18	0	10 C	10 C	5 C
19	0	0	10 T	5 T
20	0	15 C	5 C	5 T
21	0	5 C	0	0
22	0	5 T	5 T	5 T
23	0	175 T	205 T	10 T

TABLE 4—STRAINS IN MICRO-IN. PER IN. UP TO DOUBLE WORKING LOAD

Gage No.	Equivalent load, lb per ft						
	250	750	900	1050	1200	1200, held 2 days	250, unload
1	0	60 C	70 C	90 C	115 C	130 C	20 C
2	0	75 T	95 T	110 T	150 T	165 T	25 T
3	0	85 C	110 C	125 C	165 C	170 C	10 C
4	0	95 C	120 C	135 C	170 C	210 C	50 C
5	0	110 T	150 T	170 T	190 T	190 T	20 T
6	0	105 T	155 T	180 T	200 T	225 T	35 T
7	0	115 T	150 T	165 T	185 T	190 T	10 T
8	0	80 C	100 C	120 C	160 C	180 C	20 C
9	0	65 C	80 C	100 C	125 C	150 C	35 C
10	0	65 T	90 T	110 T	135 T	175 T	40 T
11	0	10 T	0	10 T	5 T	5 T	5 T
12	0	15 C	30 C	30 C	40 C	50 C	45 C
13	0	10 T	5 T	0	5 T	15 T	10 T
14	0	5 T	5 T	0	5 T	40 T	45 T
15	0	5 C	0	0	0	25 C	30 C
16	0	10 C	15 C	10 C	10 C	25 C	35 C
17	0	10 C	20 C	20 C	20 C	15 C	10 C
18	0	15 C	10 C	30 C	35 C	40 C	40 C
19	0	15 C	10 C	20 C	25 C	50 C	45 C
20	0	5 T	0	5 T	5 C	10 T	15 T
21	0	0	5 T	0	5 T	35 C	45 C
22	0	10 T	5 C	5 C	0	45 T	40 T
23	0	10 T	170 T	330 T	390 T	570 T	265 T

TABLE 5—STRAINS IN MICRO-IN. PER IN. UP TO CRACKING LOAD

Gage No.	Equivalent load, lb per ft			
	250	1200	1300	1400
1	0	120 C	130 C	155 C
2	0	145 C	155 T	195 T
3	0	160 C	185 C	210 C
4	0	155 C	170 C	205 C
5	0	190 T	205 T	245 T
6	0	180 T	200 T	275 T
7	0	175 T	190 T	240 T
8	0	150 C	170 C	200 C
9	0	125 C	135 C	160 C
10	0	150 T	160 T	200 T
11	0	0	5 C	10 C
12	0	20 C	25 C	30 C
13	0	5 T	5 C	0
14	0	5 C	0	0
15	0	0	10 T	10 C
16	0	10 C	10 C	5 C
17	0	0	0	5 C
18	0	20 C	30 C	40 C
19	0	10 C	10 C	20 C
20	0	5 C	0	10 C
21	0	5 T	10 C	10 C
22	0	5 C	10 C	10 C
23	0	740 T	880 T	2340 T*

*Indicates cracking.

TABLE 6—STRAINS IN MICRO-IN. PER IN. UP TO BREAKING LOAD

Gage No.	Equivalent load, lb per ft			
	1400	2700	4500	5550
1	—	—	—	—
2	170 T	310 T	270 T	190 T
3	240 C	450 C	860 C	1280 C
4	200 C	400 C	810 C	1220 C
5	280 T	345 T	310 T	300 T
6	265 T	355 T	350 T	375 T
7	255 T	365 T	320 T	285 T
8	230 C	455 C	825 C	1275 C
9	145 C	310 C	830 C	—
10	185 T	310 T	240 T	230 T
11	50 C	50 C	90 C	70 C
12	70 C	120 C	170 C	160 C
13	30 C	80 C	185 C	190 C
14	40 T	60 T	80 T	90 C
15	10 T	10 T	30 T	90 C
16	0	10 T	20 T	85 C
17	50 C	0	70 T	80 T
18	60 C	140 C	250 C	275 C
19	55 C	70 C	65 C	120 C
20	30 T	30 T	10 C	110 C
21	0	0	20 T	50 C
22	60 T	40 T	25 C	35 C
23	—	—	—	—

TABLE 7—END SLOPES AND DEFLECTIONS AT MIDSPAN DURING LOADING

Loading condition	Equivalent load, lb per ft											
	250	750	900	1050	1200	1200, held 2 days	1300	1400	2700	4500	5000	250, unload
Up to working load												
Deflection, in.	0	0.695										—
		0.755*										
End slope, minutes	0	4.0										0.2
		4.0*										
Up to double working load												
Deflection, in.	0	0.738	1.050	1.214	1.537	—						—
End slope, minutes	0	4.0	5.7	6.7	8.5	9.2						1.0
Up to cracking load												
Deflection, in.	0				set to zero		0.107	0.680				
End slope, minutes	0				8.7		9.7	12.7				
Up to breaking load												
Deflection, in.								1.94	5.25	15.37	25.75	
End slope, minutes								9.0	29.0	81.0	130.7	

*Held 30 minutes.

In the March 1950 JOURNAL, F. H. Jackson suggested ways to produce better pavement concrete through improved methods of construction and design of equipment. Approaching the problem from a different angle Mr. Swayze suggests finishing and curing techniques adaptable to present machinery and construction speeds and intended to improve the durability of pavements.

Finishing and Curing: A Key to Durable Concrete Surfaces*

By MYRON A. SWAYZE†

SYNOPSIS

After a comparison of past and present pavement curing and finishing techniques the significance of timing and character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

INTRODUCTION

Concern over the stability of concrete surfaces, especially those of concrete pavements exposed to frost action or to salt applications for removal of ice, is not new. Users as well as producers of cement have for at least 25 years, to the author's knowledge, been making comparisons of "old" and "new" pavements, and expressing the wish for duplication of the old type of concrete, with its highly durable surface.

In this quest for more durable pavement surfaces, it is natural that cement quality should be the first point of inquiry. Since cement is the sole binding agent for the aggregate particles, the assumption that the sole responsibility for good or bad concrete lies in the cement would seem logical. Therefore, recognized changes which have occurred in cement composition, fineness and other characteristics have for years been under the closest of scrutiny, while practices in the treatment of the fresh concrete have been given far too little attention.

The advent of air entrainment, either as an addition to cement, or as an admixture in concrete, has greatly improved resistances of exposed concrete surfaces. The 12-15 year experience with this type of concrete in severe winter climates cannot be disregarded. Neither should the warning

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of F. H. Jackson[1] on the fallacy of an assumption that "merely because we have added 4 or 5 percent air to our present day concrete, we have eliminated all of our troubles and have nothing further to worry about."

Refinements in concreting practice such as Jackson suggests—more uniform aggregate gradings, more adequate machinery for mixing and consolidating 0 to ½-in. slump concrete, and less emphasis on speed of construction—all should contribute to higher durability in concrete pavements and other exposed surfaces. However, there are other equally important means of securing resistant surfaces which need no new machinery or slowing of construction. To enumerate these and emphasize their importance is the purpose of this paper.

OLD AND NEW METHODS FOR FINISHING AND CURING CONCRETE

Concurrent with changes in cement composition and fineness which have decreased the tendency to bleed and increased the rate of strength gain, there have occurred equally important changes in methods of consolidating, finishing and curing concrete pavements. A review of old specifications covering these operations can serve to explain at least some of the reasons for superiority of "old-fashioned" cement in concrete, if they are accompanied by suitable interpretation and contrasted with modern practices. In the following quotations, italicized words and parenthetical phrases are by the author.

Wayne County, Michigan, has an enviable record in old concrete roads. Excerpts from their 1913 specifications[2] read as follows:

"The workmen shall place the concrete in position—and it shall be *well tamped*.. . . . The Contractor shall employ at least two men whose special duty it shall be to use a strike board . . . . so that the top can be properly stroked off. . . . . In all work done under the contract, all foot and other traffic . . . . must be kept off *the top of* the concrete until it has thoroughly set. . . . . After the concrete is laid, and until it is thoroughly set, it shall be protected from the sun by a canvas or other suitable covering. (Note no provision for watering.) . . . . When, in the judgment of the Engineer, the concrete is sufficiently hard to warrant, this covering shall be removed and the concrete covered with a layer of sand or gravel . . . . and sprinkled and kept damp 8 days . . . "

In Milwaukee County, Wisconsin, wetter mixes were used in roads.[2] Apparently in 1913 no surface finish was applied after the original strike-off.

"Under extreme heat or when overtaken by rain, the work is covered with canvas or other suitable means and is thus protected until it takes its initial set. After a number of hours it is covered with a light layer of earth and sprinkled each day for five days." (Here the idea of canvas was either to halt evaporation or to protect from rain.)

The Kansas Highway Department has many old roads in excellent surface condition. Their 1924 specifications for finishing and curing read in part:

"After the concrete has been deposited, it shall be leveled off from ½ to ¾ in. higher than the finished pavement and then tamped . . . . until áll voids are removed and the concrete is thoroughly compacted. The mechanical tamper shall go several times over each area . . . . Immediately after the final tamping the surface shall be belted over once . . . . When the surface of the pavement is free from water and just before the concrete obtains its initial set, the surface shall be given a final belting in order to produce a uniform finish.

"As soon as finished, the concrete shall be protected by a canvas covering, suspended not less than 12 in. above the surface, or a burlap covering laid directly on the finished surface, and *if directed*, the surface of the pavement shall be sprinkled with water. When the concrete has hardened sufficiently, the . . . . covering shall be removed, the surface of the pavement wetted thoroughly and covered with earth . . . . The earth covering (2 in.) shall be thoroughly watered twice a day for two weeks." (Ponding was an alternate to wet earth cover.)

In another Kansas specification of the same era, a hand roller was employed to compact the surface and remove water.

"The roller shall pass from one edge of the pavement to the other in one operation and rolling shall continue until free water ceases to come to the surface."

Similar finishing and curing specifications were in force in New York State in 1923. After deposition of the concrete,

"The surface shall be struck off by a steel or steel-shod template at least 6 in. wide, weighing not less than 15 lb per linear ft."

"After the concrete has been struck off and as soon as possible without causing the concrete to flow, it shall be rolled with a hand roller. (10 in. x 6 ft 0 in.; 60-80 lb wt.) The rolling shall be done from one side of the pavement. The roller shall be moved over the surface from one edge to the other in one operation. The roller shall be advanced one-half of its length, after it has passed over and back across the pavement. The rolling shall be continued at intervals of about 15 minutes, until free water ceases to come to the surface.

"After the rolling has been completed, the concrete shall be finished by a belt of wood, canvas or rubber 10-12 in. wide. . . . . After the pavement has been completed the surface shall be given a broom finish.

"The finished concrete shall be protected as follows: (a) During rainy, windy or hot weather . . . . the concrete shall be protected with canvas. Sufficient canvas to cover 100 linear ft of slab shall be provided. (b) Immediately following removal of the canvas, the concrete shall be protected with two layers of wet burlap, ... and kept wet by spraying. . . . . (c) After the burlap is removed, the pavement shall be wetted thoroughly and covered with earth, straw or other approved material . . . . and kept wet for at least 8 days."

The Ohio Department of Highways had a number of similar provisions in their 1922 specification for one course concrete pavement. The following are excerpts.

"The concrete shall be mixed in proportion of 1:1½:3. This will require 7 to 8 sacks of cement per cu yd. The concrete shall show a slump not greater than 2 in. or less than ½ in. . . . . The concrete shall be deposited . . . . in a uniform layer one in. greater than the required thickness. A mechanical finishing machine . . . . shall strike off, compact and finish the surface to the desired grade. The concrete shall then be compacted to remove all excess water.

"Immediately after the concrete has been tamped it shall be rolled with a metal hand roller (10-12-in. diameter x 6 ft: weight 60-100 lb) until free water ceases to come to the surface. All portions of the pavement shall receive at least three rollings at intervals of 15 minutes and as many more as necessary to remove excess water.

"After the rolling has been completed, the concrete shall be finished by a canvas or rubber belt or thin board. . . . . As soon as the concrete has sufficiently hardened to prevent pitting, the surface shall be sprayed with water and then covered with heavy burlap which shall be kept wet for at least 10 days." (An alternate of wet earth cover or ponding was provided after 3 days burlap cover.)

In Germany, finishing of the surfaces on the Autobahnen had to follow consolidation of the concrete rather closely, on account of the very dry,

stiff mix which was used. However, following this operation, there was a long interval when no water was lost or gained by the hardening concrete, since it was protected from sun, wind or rain by a series of low-lying canvas tents. Moist curing mats were not applied until 5-6 hours after the concrete was placed and thorough soaking of the surface did not begin until after 24 hours. Water curing by wet earth or ponding was then meticulously carried out. Durability of their surfaces is excellent.

Contrasted with these old practices in finishing and curing concrete surfaces, nearly all modern methods tend to hurry both the application of the final finish, and the covering of the surface with wet burlap, cotton mats, paper or liquid curing agents which are impervious to moisture. Curing after the 24-hour period is highly variable from one state to another, ranging from no curing at all to varying periods of water application. The reader may write his own ticket, depending on from which state he hails.

SIGNIFICANCE OF TIMING AND CHARACTER OF FINISHING

T. C. Powers in his 1939 report on bleeding[3] gives an excellent picture of what happens in a newly-laid concrete slab up to the beginning of set. Sedimentation gradually builds up a zone of maximum compaction at the base of the slab, a transition zone of variable but increasing water content above, and a third zone with constant and highest water content near the surface, where only clear water is found. Depending on the relative rates of settlement and set, any one of these zones may be "frozen" at the surface by the stiffening of the concrete due to setting. It should be obvious that any action by machinery used for consolidation of the fresh concrete which tends either to hasten the upward flow of water to the surface or to delay the stiffening of the concrete due to set will have a beneficial action, by lowering the water-cement ratio at the surface. The old-fashioned practice of rolling and rerolling the surface during the later part of the bleeding period accomplished this result. The same effect probably could be induced by a re-use of the modern longitudinal screed or "bull-float" at a time when stiffening of the concrete has just begun. Removal of this liberated water from the slab surface is of course essential.

Both the timing and character of the final finishing of pavement surfaces need reconsideration and study. L. S. Brown suggested in 1940[4] that a good cement paste is resistant to frost action because the capillaries in the paste are so small that water in these spaces does not freeze at the temperatures imposed. T. C. Powers has more recently confirmed this deduction, and has defined such a paste as one with a water-cement ratio of 4.5 gal. per sack or less.

Applying these deductions to field practice, it should be evident: first, that the concrete mix should be so designed as to carry a minimum of mixing water; second, that as much of this mixing water as possible be removed from the concrete before set; and last, that just prior to set, the top layer of

the pavement be compacted mechanically as much as possible without destroying the hydration structure already set up.

Evidences of the benefits of late compaction of pavement surfaces are abundant, if we will but observe and profit by them. Tire marks and foot prints in pavement surfaces made considerably after the final "finish" was applied are invariably harder and more durable than the rest of the surface. Retroweled areas on sidewalks are notable in two respects; their outstanding hardness and durability in comparison with other areas, and their darker color, which is evidence of greater density, and hence, lower light reflectance.

All of this adds up to the suggestion that nearly all modern finishing of concrete surfaces is performed too soon—the operation should be delayed until the concrete is close to its initial set, and should be immediately preceded by a compacting of the upper portion of the slab.

SIGNIFICANCE OF TIMING AND MODE OF EARLY CURING

The numerous methods for the protection and early curing of concrete surfaces, and the widely variant types of covering which are employed during the first 24 hours—wet earth, burlap, cotton mats (wet, dry or soaking), waterproof paper and membrane curing solutions and compounds—inevitably raise the question: What is the *best* way of conducting the early curing of concrete, to promote high durability in the surface? To appraise these methods and to arrive at a logical answer to this question, an examination of the needs for early protection is in order.

The primary function of any type of cover for fresh concrete is to prevent evaporation of water from the surface, which would result in a shrinkage in volume and the formation of shrinkage cracks in the surface. No added water is needed to prevent these cracks: concrete cured under waterproof paper or various membranes is as free from them as under the wettest of burlap or cotton mats. As to timing, cover is needed the moment the surface has reached its initial set. Prior to this the concrete is plastic, free water can be brought to the surface by working or slight pressure, and evaporation merely reduces the water-cement ratio.

Following the arrival of initial set, the capillary system in the cement paste develops rapidly. As pointed out by the author in a previous paper[5] the forces of capillary action and surface tension will have a strong compacting action in themselves on the surface, if no further water is added at this stage in hardening. They will also tend to equalize differences in water content between the top and bottom of the slab, set up by sedimentation during the earlier periods. On the other hand, early application of curing water prevents surface compaction by breaking the surface tension, and also maintains the inequalities that exist in water content of the concrete. In addition, there is always danger of absorption of organic matter from moldy covering if water is added through such material.

Another function of covering during the first 24 hours is protection of the surface from the CO_2 in the atmosphere. L. S. Brown[4] observed that

surface carbonation takes place quickly with fresh pastes, but is exceedingly slow on surfaces exposed after more or less extended curing. Carbonation affects all of the lime in the paste, whether present as hydroxide or within the silicate gel. Fragments of scale from roads always had a relatively open structure, and lime in them was completely carbonated. His conclusion was that the rate of penetration of CO_2 was a function of pore size, hence the need for exclusion of the atmosphere during the curing period until hydration was well advanced.

All of these observations point to the conclusion that early application of curing water is wrong, but protection of the surface from evaporation and from carbonation by contact with the air is essential. The duration of this early curing stage is problematic; it probably should end at the time when the concrete has reached its maximum temperature and is beginning to cool. At this time, addition of plenty of curing water will greatly reduce the shrinkage due to the loss of heat by inducing a compensating expansion by absorption of water.[5] If the concrete is already saturated by early water additions, no such compensation is possible.

NEED FOR LATE WATER CURING

Besides this compensation for thermal shrinkage by the late addition of curing water, maintenance of the surface in a wet state has other advantages. Hydration of cement ceases when the surface is allowed to dry and due to shrinkage of the gel already formed around unhydrated grains, is not easily started again on reapplication of new water. Therefore, wet curing once begun should be continuous until the permanent hydration structure is well developed.

Membrane curing compounds serve admirably during early stages of curing in preventing formation of surface cracks, if they are applied in sufficient quantity to hold the water in the concrete. However, this very efficiency in the early stages is equally effective in keeping out later applications of needed curing water. Comparisons of surface structures as set up under membranes and under wet curing are badly needed. In the author's opinion, the method should be held in serious question unless it can be demonstrated that it can produce equal hydration of cement grains and equal density of the hydrated paste at the surfaces of slabs so "cured." In a comparative test of wet versus membrane curing in Kansas in 1949, the author observed a softer surface in membrane-cured sections than in water-cured areas, after six months of surface use.

The duration of needed late water curing is difficult to define in terms that may be written into a specification. The time is intimately tied to the rate of hydration of the cement, which in turn is influenced by the temperature at which the concrete is cured and by the composition and fineness of the cement itself. Under equal temperature conditions, cements of Type I and II will require more curing than Type III products, and perhaps

less than portland-pozzolans, some of which at least are slow in early rates of hydration.

Since hydration and strength development generally progress hand-in-hand, probably the best way to define duration of late water curing is on the basis of attainment of minimum strengths of specimens cured at the job under identical conditions as the road surface. In the case of beam tests, the top surface is suggested as the one to subject to tensile stress.

LABORATORY TESTS—EFFECT OF TIME OF FINISHING AND CURING

Fig. 1-4 show the relative durabilities of four cements in six-sack concrete mixes after 100 cycles exposure to freezing and thawing with calcium chloride application to the surface. All four cements were manufactured at the same plant. They consisted of a pair of Type I and IA cements and a pair of Type III and IIIA cements. In each figure, the three slabs vertically were cast from the same batch of concrete, but finished and cured differently for the first 24 hours. The pairs of slabs horizontally were from identical concrete batches mixed on the same day and finished and cured identically.

The top pairs of slabs were broomed as soon as the surface would hold broom marks, covered a half hour later with wet burlap and kept wet for 24 hours. The center pairs were broomed just before initial set and covered immediately with cellophane to prevent either gain or loss of water for 24 hours. The bottom pairs were steel-troweled just prior to initial set to compact the surface, then broomed and covered with cellophane for 24 hours. At the age of 24 hours all slabs were removed from the molds, provided with a $\frac{3}{8}$-in. dam around the top edge and placed in the fog room for wet curing. All top surfaces quickly became covered with water. The Type I and IA concretes were given 6 days wet curing and 21 days dry storage at 50 percent

Broomed at 1 hour
Wet burlap 1 hour 30 minutes-24 hours

Broomed at 3 hours
Cellophane cover 3 hours 5 minutes-24 hours

Steel troweled at 3 hours 15 minutes
Broomed at 3 hours 20 minutes
Cellophane cover 3 hours 20 minutes-24 hours

1st Batch 2nd Batch

Fig. 1—Condition of Type I cement, 6-sack concrete slabs after 100 freeze-thaw cycles. Treatment after 24 hours—Six days ponded water curing followed by 21 days in air at 50 percent relative humidity. The top pair, which received early water curing, are covered with progressive scale and are close to disintegration. Scaling of the other specimens, which received no added water for 24 hours, is not progressive.

Broomed at 1 hour 25 minutes
Wet burlap 1 hour 30 minutes-24 hours

Broomed at 2 hours 45 minutes
Cellophane cover 2 hours 45 minutes-24 hours

Steel troweled at 3 hours 5 minutes
Broomed at 3 hours 10 minutes
Cellophane cover 3 hours 10 minutes-24 hours

1st Batch 2nd Batch

Fig. 2—Condition of Type IA air-entraining cement, 6-sack concrete slabs after 100 freeze-thaw cycles. Treatment after 24 hours—Six days ponded water curing followed by 21 days in air at 50 percent relative humidity. Scaling of the upper pair which received early water curing is about equal to that on the middle pair which received delayed finishing and no water for 24 hours. None of the scaling is progressive. The bottom pair, which were troweled, broomed and covered at 3 hours are perfect or nearly so.

relative humidity before subjection to the first freezing cycle. The Type III and IIIA concretes were wet-cured 2 days and kept in dry storage for 11 days before freezing.

Freezing was done in a room at −20 F, with ¼ in. of water on top of specimens. They were held overnight at this temperature and, on removal next morning, flake calcium chloride was sprinkled over the ice surface in the concentration of 2.5 lb per sq yd of concrete. In the afternoon after thawing was complete, the slabs were washed off with a hose, dams refilled with fresh water and specimens returned to the freezer room.

Broomed at 55 minutes
Wet burlap 1 hour 25 minutes-24 hours

Broomed at 1 hour 40 minutes
Cellophane cover 1 hour 45 minutes-24 hours

Steel troweled at 2 hours 5 minutes
Broomed at 2 hours 10 minutes
Cellophane cover 2 hours 10 minutes-24 hours

1st Batch 2nd Batch

Fig. 3—Condition of Type III cement, 6-sack concrete slabs after 100 freeze-thaw cycles. Treatment after 24 hours—Two days ponded water curing followed by 11 days in air at 50 percent relative humidity. Scaling on the top pair of specimens is approaching the progressive stage. The center and bottom pairs are still in good condition, with the bottom pair in best shape.

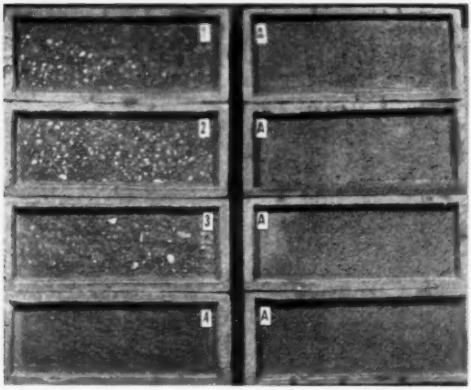
Fig. 4—Condition of Type IIIA air-entraining cement, 6-sack concrete slabs after 100 freeze-thaw cycles. Treatment after 24 hours—Two days ponded water curing followed by 11 days in air at 50 percent relative humidity. Scaling is moderate on all specimens but least with the compacted surface and no water application for 24 hours.

These tests, which were a pilot series, gave so good an indication of the value of some compaction of the surface during finishing and of delayed application of curing water, that a second series was run on three cements used in the 1949 Kansas Test Road south of Topeka. These three cements, together with a modern Type IA air-entraining cement from the same plant, appear in Fig. 5-11 in the following order from top to bottom of the photographs: (1) modern Type I portland cement (2) modern clinker coarse ground (1400 SA), (3) "old fashioned" cement (1924 specifications) and (4) modern air-entraining Type IA cement.

Each pair of specimens from left to right was cast from the same batch of concrete, but finished and cured differently. All specimens were given a preliminary finish with a wood float 20 minutes after casting. The left hand slabs were then allowed to stand uncovered until 2 hours to 2 hours 20 minutes old, then broomed and covered with a wet mat 20 minutes later. The mats were kept thoroughly wet for 24 hours. The companion slabs at the right were rolled lightly at half hour intervals, beginning at the age of 1½ hours, to simulate an old Kansas Highway specification. Some water was brought to the surface and rolled off even with the air-entraining concrete, except that the surface of this type of concrete became too sticky for further rolling after the third working. The other three concretes were rolled four times.

The final finishing consisted of a light steel troweling followed by sawing of a wood float across the surface just before initial set of the various concretes. A cellophane cover was placed over the molds 20 to 30 minutes later, and this in turn was covered with a wet blanket to prevent either loss or gain of water by the slabs. This curing continued for 24 hours.

At this age all slabs were removed from molds, provided with a ⅜-in. dam around the edges, and then subjected to wet storage for 21 days. Following the water curing, all specimens were stored for 28 days longer in dry storage



1949 Specification cement

1949 Specification clinker, coarse ground

1924 Specification cement

1949 Specification air-entraining cement

Early brooming, wet burlap cover

Rolling with delayed finish, cellophane cover 4-24 hours

Fig. 5—Condition of first round of test specimens made with four different cements prior to first freezing cycle

at 55 to 60 percent relative humidity. The freezing and thawing cycles were then conducted as in the previous pilot series.

Fig. 5 shows the condition of the first round of test specimens prior to the first freezing cycle. The scaled appearance of the upper three slabs at the left was caused by the application of water through a burlap cover. The set of the top skin of cement was retarded to such an extent that it was rubbed off when the slabs were washed prior to casting the dams around the edges.

1949 Specification cement

1949 Specification clinker, coarse ground

1924 Specification cement

1949 Specification air-entraining cement

Early brooming, wet cotton cover

Rolling with delayed finish, cellophane cover 4-24 hours

Fig. 6—Second round of test specimens made with four different cements before beginning of freezing test

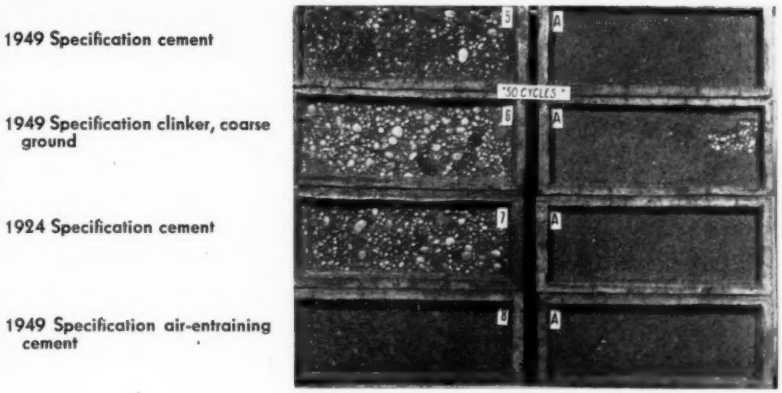

1949 Specification cement
1949 Specification clinker, coarse ground
1924 Specification cement
1949 Specification air-entraining cement
"50 CYCLES"
Early brooming, wet cotton cover
Rolling with delayed finish, cellophane cover 4-24 hours

Fig. 7—Second round specimens after 50 freeze-thaw cycles

This picture is shown to illustrate a condition which may occur on highway jobs when the field force is over-zealous in soaking burlap cover at early ages. Such an occurrence has already been reported by the writer.[5]

Fig. 6-11 illustrate the progressive deterioration of concretes subjected to early finishing and water curing as freezing and thawing progressed from 0 cycles (Fig. 6) through 50 cycle intervals up to 250 cycles (Fig. 11). The slabs shown are those made in the second round of tests. In this round as in the third (not shown), early water curing for slabs at the left was under

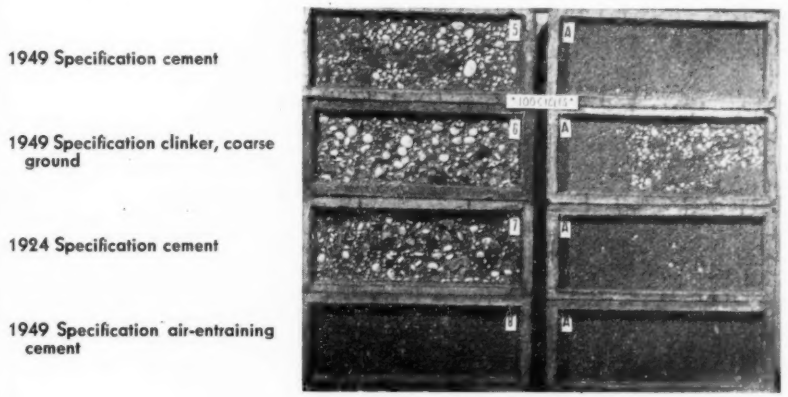

1949 Specification cement
1949 Specification clinker, coarse ground
1924 Specification cement
1949 Specification air-entraining cement
"100 CYCLES"
Early brooming, wet cotton cover
Rolling with delayed finish, cellophane cover 4-24 hours

Fig. 8—Second round specimens after 100 freeze-thaw cycles

1949 Specification cement

1949 Specification clinker, coarse ground

1924 Specification cement

1949 Specification air-entraining cement

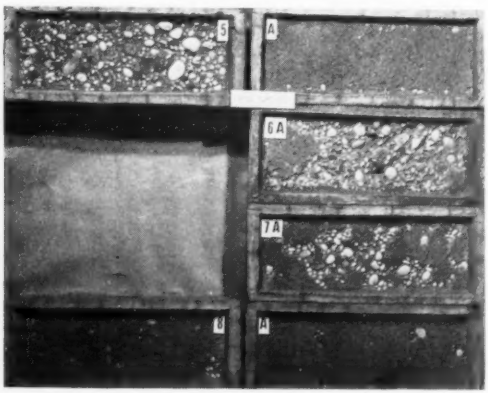
Early brooming, wet cotton cover

Rolling with delayed finish, cellophane cover 4-24 hours

Fig. 9—Second round specimens after 150 freeze-thaw cycles

wet cotton pads, which did not affect the hardening of the surface. Gaps which show in succeeding photographs are slabs which were too badly softened by frost and salt action to hold water in either old or new dams around the edges. Slabs 6 and 7 were lost at 100 cycles, Slab 5 at 200 and Slabs 6A and 7A at 230 cycles, at which times the surface disintegration shown had extended deep into the specimens.

From comparison of these successive figures, several points are evident. First, the air-entraining cement had outstanding resistance. However,



1949 Specification cement

1949 Specification clinker, coarse ground

1924 Specification cement

1949 Specification air-entraining cement

Early brooming, wet cotton cover

Rolling with delayed finish, cellophane cover 4-24 hours

Fig. 10—Second round specimens after 200 freeze-thaw cycles

Fig. 11—Survivors of 250 freeze-thaw cycles—all three rounds

even this type of concrete is improved by compaction of the surface, late finishing and late application of curing water. Second, reduction in fineness of cement or in both lime content and fineness (coarse ground and old-fashioned samples, respectively) produced no increase in durability when all of the concretes were finished and cured in the same manner; to the contrary, both coarse cements showed lower resistance than the modern plain cement of normal fineness when cured under identical conditions. Third, the old-fashioned techniques of rolling and finishing, combined with delayed application of curing water, materially extended the life of each type of concrete, regardless of the type of cement it contained, in comparison with one modern method of finishing and curing.

Our conclusions from these tests, and from consideration of other points discussed in this paper, are that old-fashioned cement *per se* furnishes no panacea for the ills of concrete. Its sole virtue, if it can be called such, very probably lay in a much greater tendency to bleed which, combined with a slightly longer setting time, forced the contractor to delay his finishing operations until this bleeding water was out of the way. Paving crews of 25-30 years ago were well spaced out behind the mixer. Even where early wet curing was specified, the covering crews were so far behind the mixer in time and space that insistence on the strict performance of their duties was frequently neglected. Late water curing was quite generally the rule, with benefit to the stability of the pavement.

This loss of water by bleeding unquestionably lowered the water-cement ratio of the concrete surface. However, the loss of such water was rarely more than 10 percent of the total mixing water. This same reduction in mixing water can be obtained now with air-entraining cement, if paving

mixes are redesigned to compensate for the air entrained and the increased workability.

As portland cements have improved over the past 20-25 years in strength-gaining characteristics and relative freedom from bleeding, paving contractors in general have taken advantage of these properties by speeding up not only their rate of placing concrete, but also the timing of subsequent operations. Since there has been little or no study of the effect of either the character or the time of finishing and curing operations on concrete surface durability, many specifications now in force are either vague on these points, or else are insistent on what this author believes is wrong procedure for the best resistance in concrete surfaces.

It is most earnestly recommended to all who are responsible for the production of durable concrete surfaces that less emphasis be placed on relatively slight differences in cement compositions, and far more on the treatment the cement receives when mixed into concrete. Not that the physical properties of cement as revealed by performance tests are unimportant—they are—but that so much more is to be gained by strict attention to early methods of surface treatment of the concrete made from it.

SUMMARY

Much needs to be done in the way of laboratory studies to evaluate differences in finishing and curing methods, and effects of different methods on surface durability. However, the following principles, many of which were once in use, can well be applied now, without waiting the outcome of laboratory investigations.

1. Make all concrete which is exposed to the deteriorating action of frost air-entraining concrete, either by use of air entraining additions to the cement or by addition of similar admixtures at the mixer. Redesign concrete batches to take advantage of the lower sand requirements and lower water-cement ratios that go hand-in-hand with air entrainment. Establish cement factors not on the basis of attaining minimum strengths, but on reasonable water-cement ratios that are in the range of high durability.

2. Use the least amount of mixing water compatible with first, good mixing and second, the ability of subsequent machinery to consolidate the slab.

3. After the concrete is placed on the subgrade, thoroughly compact it and bring the surface to the desired level quickly, while the concrete still has its maximum workability. Space out later operations so that they will perform their required function at the proper time, not by the watch, but in relation to the setting rate of the concrete as influenced by air temperature, sun, humidity and wind.

4. Allow water to evaporate from the fresh surface at any reasonable rate—it can be excessive in hot, sunny weather with a dry wind, when a dry awning riding the forms would prevent excessive loss. In less rapid drying weather, if water comes to the surface or can be induced to do so, get rid of it—it serves no useful purpose at any time prior to the initial set of the concrete.

5. While the surface is still somewhat plastic, use hand straightedges to smooth out high and low spots, and not to put a 6 or 8 ft dent in the surface to prove to posterity that an inspector had tried the surface for trueness.

6. Just prior to attainment of initial set in the surface, compact it with either a rotary power steel float or light longitudinal screed. Follow this operation immediately with the final surface finishing operation.

7. Cover the pavement immediately after this finish has been applied, but do not add new water to it at this stage. The purpose of cover at this time it to protect against evaporation and shrinkage cracks and to prevent carbonation of the surface layer. Burlap and cotton mats need occasional moistening, but the concrete beneath still has all the water it needs for cement hydration—do not increase the supply. Maintain this condition at least until the concrete has reached its maximum temperature and has begun to cool.

8. Following this first curing stage of protection but no watering, wet the slabs immediately after the initial cover is removed, and follow this wetting quickly with wet earth, ponding or other means of saturating the concrete. Maintain this wet condition until the pavement has reached service strength.

In all of the above suggestions, there need be no loss in the productive rate of laying modern concrete pavements. What is needed most is not new machinery, but merely proper timing with what we have, and the realization that water-curing can be started too early in the life of the concrete. The sooner we recognize that this whole life is influenced by what we do to surfaces in the first few hours, the sooner will come fulfillment of the "Creed of a Concrete Man":[6]

"Concrete is my business. My future welfare and the well-being of my family depend on the quality of the work I do. I have chosen concrete out of all the construction materials of the earth as the one in which to specialize. I have done this because I feel that concrete is a material of integrity. Mixed and placed with understanding hands, concrete will not fail. Therefore, the work I do well now will survive and serve long after I am gone."

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6. Reprint from "The Concrete Builder," courtesy Portland Cement Assn.

Discussion of a paper by Myron A. Swayze:

Finishing and Curing: A Key to Durable Concrete Surfaces*

By T. C. POWERS, T. W. THOMAS and AUTHOR

By T. C. POWERS†

Mr. Swayze's discussion of finishing and curing pavements deserves careful attention by all who are in any way responsible for the quality of concrete pavements. Certainly his arguments are well enough supported to warrant a critical re-evaluation of current practices. We need also to keep in mind Mr. Jackson's conclusions regarding the German Autobahnen.

In a 1942 ACI paper‡ Mr. Swayze emphasized the advantages of withholding curing water from the concrete during the period of temperature rise and adding it during the period of falling temperature. In the present paper he not only refers to the advantage of so modifying thermal volume changes but also deals with the extra consolidation that might be produced if curing water is withheld during the time while the concrete is plastic. I should like to take this opportunity to augment Mr. Swayze's discussion of phenomena occurring during the period of plasticity.

Mr. Swayze mentioned the different zones of compaction that develop in a layer of concrete during the period of bleeding, particularly the compression zone. We may think of the compression zone as a consolidated region that builds up from the bottom at a rate determined by the rate of bleeding and the bleeding capacity.§ Forces applied at the top will produce another compaction zone that begins at the top and gradually develops downward if the force is sustained. Such consolidation is, of course, accompanied by the displacement of water. Mr. Swayze emphasizes the advantages to be gained by the intelligent application of such forces, saying, "as much of the mixing water as possible should be removed from the concrete before set."

Half measures will not do. If the process of consolidation is begun and ended too soon, the effect may be detrimental even though some water might have been removed from the slab that would otherwise have remained in it. In other words, beginning and ending the finishing process too soon can produce a slab that will scale under frost action, the scaling actually being the indirect result of the reduction in water content.

*ACI JOURNAL, Dec. 1950, *Proc.* V. 47, p. 317. Disc. 47-22 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 4, Dec. 1951, *Proceedings* V. 47,

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‡Swayze, M. A., "Early Concrete Volume Changes and Their Control," ACI JOURNAL, Apr. 1942, *Proc.* V. 38, pp. 425-440.

§PCA *Bulletin* 2, p. 84.

This effect is understandable if one keeps in mind the factors that control the rate of bleeding. The rate of downward movement of solids in fresh concrete, and hence the rate at which water appears at the surface, is determined by the weight of the solids and the impedance to flow of water through the spaces between the solid particles. The principal variable is the impedance to flow. The magnitude of this impedance is governed mainly by the ratio of water to the total surface area of all the solids in the mix. If the ratio of water to surface is decreased, the resistance to flow increases and the rate of bleeding decreases. When the rate of flow is equal to or less than the rate of evaporation, we observe that the mix is "not bleeding." Actually the surfaces of all plastic mixes must subside to some degree, unless the normal processes are arrested by premature stiffening. In general the differences between bleeding and nonbleeding mixes as judged in the field is a matter of degree rather than kind, and it is partly a matter of weather.

After fresh concrete is placed on the subgrade, the ratio of water to total surface can be changed by applying forces at the surface. However, forces applied by means of screeds, floats, trowels, etc., reduce the water content progressively from the surface downward. The initial effect is to squeeze the water out of the topmost layer. During this process little or no consolidation of the underlying regions of the mass is produced because the solid particles are mostly not in contact with each other; the forces produced below the topmost layer are mostly hydrostatic. Therefore, a brief manipulation early in the process of settlement reduces the water-to-surface ratio in the top layer only.

Consequently, the top layer tends to settle through the water at a lower rate than the material just below it. When settlement continues after finishing is stopped, the consolidated top layer tends to lag behind the underlying material "floating" in the mixing water. Thus a plane (or more probably a zone) of weakness may develop under the surface layer.

This kind of manipulation, not properly timed with respect to the bleeding process, is, as was pointed out in 1939,* probably a common cause of surface scaling. As already mentioned, this undesirable effect is to be expected even though the manipulation that produced it caused a reduction in the total water content of the slab.

If manipulation is repeated from time to time, or carried on continuously, until the whole slab is consolidated to whatever degree is possible from the combined effects of gravity and manipulation, the conditions described above can be avoided and, as emphasized by Mr. Swayze, the whole operation can be beneficial.

To produce a significant degree of consolidation it is necessary to apply a force large enough to do the work within the period while the paste remains plastic. Moreover, the force must be applied in such a way that the solids can be moved downward while the displaced water is free to escape upward.

*PCA *Bulletin* 2, p. 124.

This requirement is fulfilled when using filter mats and vacuum pumps for applying atmospheric pressure; pressures up to 2000 lb per sq ft can be produced in this manner. Mr. Swayze contends that less drastic procedures may be effective, if properly managed.

Mr. Swayze emphasizes the importance of capillary forces. These forces may be comparable in magnitude and effect with the more familiar mechanically applied forces, or they may not exist at all, depending on how the finishing and curing schedule is managed. We can account not only for the existence of these forces but also we can calculate with some degree of assurance their possible intensities.

As a starting point we bring back to mind the fact that bleeding is caused by the downward movement of the initially slightly separated solid particles in the concrete, which downward movement displaces water upward. Several factors tend to prevent the appearance of this displaced water at the surface: one is evaporation caused by sun and wind, already mentioned; another is internal absorption due to chemical processes (self-desiccation); another may be absorption by partially dry aggregate; still another may be absorption by an unsaturated subgrade. Whatever the causal factors might be, if the rate of bleeding becomes less than the rate of water loss from the surface, we observe that the surface loses its sheen. The disappearance of sheen is evidence that the plane, free-water surface has become a series of curved surfaces. Each of the curved surfaces is trying to become plane and this tendency produces an upward force. The reaction of this force is carried by the solid particles to which the curved water-surfaces are attached, and this reaction is of course a force directed downward. The magnitude of this downward force will depend upon the curvature of the water-surface. The curvature of the water-surface depends on the geometry of the interparticle regions and the degree to which the water fills these regions in the surface layer.

Carman* has shown that the maximum possible capillary tension in a water-filled granular bed occurs just as the water-level drops below the upper boundary of the bed. The magnitude of this force can be calculated from the following relationship, which is adapted from the basic relationships given by Carman:

$$P = \frac{\lambda S}{w} = \frac{\lambda S/c}{w/c} \qquad (1)$$

where P = pressure difference across the water surface
λ = surface tension of water = 73 dynes per cm at room temperature
S = the total surface in the mix
w = volume of water in the mix
c = weight of cement in the mix

Since the surface area of the aggregate in the mix is negligible, we may write,

$$S = c\, v_c\, \sigma_c \qquad (2)$$

Soil Science, V. 52, 1941, pp. 1-14.

where c = weight of cement in the mix
v_c = specific volume of cement, cc per gram
σ_c = specific surface of the cement, sq cm per cc of cement

Hence, from (1) and (2)

$$P = \frac{\lambda v_c \sigma_c}{w/c} \quad \ldots\ldots (3)$$

For a typical cement

$\sigma_c = 10^4$ sq cm per cc (air-permeability method)
= 1800 sq cm per g (Wagner)
$v_c = 0.317$ cc per g

Then if $w/c = 0.5$ and $\lambda = 73$ dynes per cm

$$P = \frac{73 \times 0.317 \times 10^4}{0.5}$$
$$= 0.46 \times 10^6 \text{ dynes per sq cm}$$
$$= 6.7 \text{ psi}$$
$$= 960 \text{ lb per sq ft}$$

From similar calculations we find that at the same water-cement ratio, P would be about 500 lb per sq ft for a coarse cement (1200 Wagner), and about 1400 lb per sq ft for a fine cement (2500 Wagner).

Thus we find that the maximum possible pressure due to capillary forces at the surface of a mass of fresh concrete is directly proportional to the specific surface of the cement and inversely proportional to the water-cement ratio. The maximum possible pressures may be expected to lie between 500 and 1400 lb per sq ft.

It is interesting to compare the magnitude of the forces with which we are dealing. The force due to gravitation alone, that which causes normal bleeding, amounts to about 100 lb per sq ft, maximum. Hence, according to our calculations showing that intensities of 500 to 1400 lb per sq ft are added when the maximum capillary force comes into play, the rate of bleeding can be perhaps 6 to 15 times the normal rate. We should expect that if the capillary forces persist until after the time when the compression zone reaches the surface, the ultimate degree of compaction of the compression zone will exceed that produced by gravity alone. Moreover, the added compaction will probably be greatest at the top, where it is most needed.

A part of Mr. Swayze's discussion has to do with control of those factors that determine whether or not capillary forces will develop. The first requirement is that the rate of evaporation must exceed the normal rate of bleeding. When the opposite is true, *i.e.*, when the rate of bleeding exceeds the rate of evaporation or when water is added to the surface by the road builder, no capillary force will be developed, regardless of the characteristics of the mix. On the other hand, if the rate of bleeding is less than the rate of evaporation and curing water is withheld, the water surface will tend to drop below the apparent surface of the concrete and capillary force will be produced. This force acts on the topmost particles and adds to the gravitational force already acting on those particles. It therefore increases the rate of settlement of the

topmost "layer" and hence the rate of bleeding. Therefore, a steady state will automatically be produced in which the rate of bleeding will exactly equal the rate of evaporation. (Of course, this would not be true if the rate of evaporation ever exceeded the rate of settlement under the maximum possible capillary force indicated by Eq. 3.)

Those factors that produce high capillary pressures (high specific surface and low water-cement ratio) are the same factors that produce relatively low rates of bleeding. With very coarse cements the capillary force is likely to be zero, while it is likely to be some finite value, probably less than the maximum indicated by Eq. 3, when using fine cements.

The importance of properly managing the finishing and curing schedule stands out when considering capillary forces, just as it did when considering mechanical compaction. The control of capillary forces is at least as subject to abuse as the application of the ordinary tools in finishing and curing concrete. The quality of the result should depend on the time at which the capillary force begins to develop and upon the length of time it is maintained, just as it does for mechanical processes. If the surface dries during the bleeding period, compaction of the topmost layer begins at once, and the surface zone of compaction slowly deepens with time. If the process is interrupted by applying curing water, the effect could be like that of beginning and ending trowelling too soon.

By T. W. THOMAS*

Mr. Swayze presents an able case in defense of modern cements and his findings for air-entraining cement are in agreement with research laboratories throughout the country. His emphasis upon the vital importance of curing cannot be questioned. There remains unanswered, however, the challenge that concrete paving equipment is primarily designed and operated to produce a smooth riding surface rather than a durable one. The sight of a longitudinal float pushing a slurry back and forth across a slab is one to cause apprehension in the mind of even the most optimistic of observers. While the compaction of laboratory specimens by means of a steel trowel shows beneficial results, it is doubtful if a related process is practical in the field. The idea of delayed finishing connotes over-finishing, reworking, and retempering, with their detrimental effect on durability. It is a matter of common observation that pavement disintegration is more prevalent at joints than at cracks. While the disintegration in each case is associated with the entrance of water, the greater disintegration at the joints is believed to be associated with over-finishing or excessive working. It is possible that the vacuum process could be modified so that rigid interlocking vacuum plates could provide a true riding surface through the action of their compaction pressure, but this is a costly process.

With our emphasis upon water-cement ratio, we have perhaps failed to put sufficient emphasis upon the water *content* of concrete. It is only through

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a reduction of the water content that an appreciable reduction in shrinkage and its related ills can be accomplished. It is suggested that the methods of construction utilized in soil cement be applied to concrete. Water contents of 190 lb per cu yd would be possible in contrast to the present 260, with a resultant 60 percent reduction in shrinkage. Such a procedure could include central mixing (possibly a pugmill type), transportation in non-agitating trucks, steel forms, a power screed, and compaction by sheepsfoot rollers. Water curing would be required since membrane cure is not desirable on an unsaturated mixture. A centerline joint would be needed, but contraction joints could be eliminated. Such a pavement would, of course, require a surface, such as a high-type bituminous pavement. The reduction in cost possible with this type of construction should do much to offset the added cost of the surface. The reduction in water content would either provide a greater strength or permit a reduction in cement content.

AUTHOR'S CLOSURE

The author is appreciative of the discussions of his paper offered by Mr. Powers and Professor Thomas. Mr. Powers has demonstrated mathematically that a compacting force will exist at the top surface of freshly laid concrete, provided that the level of the water in the concrete is very slightly below that surface. The intensity of this force, due solely to fineness of the unhydrated cement, is shown to be 500 lb per sq ft for coarse cements (1200 Wagner surface) and 1400 lb per sq ft for fine cement (2500 Wagner surface). This compacting force will continue to be exerted as long as the water surface remains below the solid surface.

While forces in this range of magnitude are not negligible, they are insignificant in comparison with those which develop as soon as the cement begins to form a hydration structure. This gel formation has a fineness very much higher than the cement itself, and the capillaries in the paste, which are the source of the compacting force, decrease rapidly in diameter as they are filled by this gel. At the age of 12–14 hours the force exerted on the unwet surface of the hardening concrete may easily reach pressures of the same order per square inch as Powers has calculated for a square foot. (See section on autogenous shrinkage, in "Early Concrete Volume Changes and their Control," ACI JOURNAL, Apr. 1942, by present author.)

These very high forces of compaction begin to be exerted after the bleeding stage is over. Their intensity very probably equals or exceeds the compressive strength of the concrete at the same early ages. Since mixing water is now being rapidly chemically combined and absorbed by the gel, the presence of this compacting force, whenever the surface has no free water, explains early shrinkage observable in concrete. This shrinkage about balances the expansion in fresh concrete caused by heat liberation of the hydrating cement, and gives added reason for the withholding of curing water until the concrete has reached its maximum temperature and has begun to cool. Application of plenty of curing water at that time will then offset the shrinkage

due to loss of heat, and will materially lessen the tendency of the concrete to crack from the stresses set up by thermal shrinkage.

Both Powers and Thomas have pointed out the dangers of over-manipulation of the surface which affect surface durability. These are valid criticisms. The author agrees that no surface should be so worked. Bleeding of highway concrete with modern normal cements is much less than with the old-fashioned coarse-ground products. With air entrained it is practically nonexistent. However, on jobs where bleeding does occur, use of the longitudinal bull float should be timed to catch it at its peak and remove it. Operation of the float should be so managed as to throw the water over the side of the forms, instead of sawing it back and forth across the slab. Many float operators have a disturbing habit of raising the float at each end of its travel, thus producing the effect of which Professor Thomas complains. This practice should be prohibited. A similar ban should be placed on use of hand edging tools at joints and edges of slabs after set has taken place in the concrete. The author has actually seen partially hardened concrete crumbled by poorly instructed finishers in their efforts to produce a nice, smooth slab edge. The damage thus done is usually irreparable, since it is hidden.

Professor Thomas expresses doubt over the practicality of steel troweling a pavement surface just before the incidence of initial set. The author calls his attention to the fact that there are several makes of power-driven rotary troweling machines which are extensively used in producing a hard finish on heavy duty floors. Besides their material advantage in labor saving, the density and uniformity of the surfaces they produce is much superior to those obtained by hand troweling. While these troweling machines have also been used in finishing pavements to some extent, such use is not as widespread as the benefit it affords should indicate.

Concerning this mechanical compaction of the concrete surface just before arrival of initial set in the concrete, the question naturally arises as to its necessity, when, by waiting a little longer the natural forces of compaction will greatly exceed in pressure any that may be mechanically applied. The answer to this lies in the fact that in all concrete, whether there has been visible bleeding or not, there are innumerable water channels of appreciable diameters leading downward from the top surface. If these are left undisturbed, the water in them is soon absorbed by the stiffening concrete, and the air "pipes" which remain allow the natural compacting forces to work from within as well as from the surface of the concrete. Frequently, small shrinkage cracks can be found which started in one of these vertical pipes.

On the other hand, if the surface is troweled at the time these major capillaries are being emptied of water by absorption, they can either be collapsed or filled with paste by application of only moderate pressure. A steel float is preferable to a wood float for this work, as it is less apt to disturb the bedding of the larger grains of aggregate. The timing for this operation is not critical, but must be after the stage when the pipes are full of water and before the

arrival of initial set. Usually there are 20 to 30 minutes of elapsed time between these two points in the progress of cement hydration.

With the major water channels in the top section of the slab either collapsed or filled with grout by the trowelling, the later shrinkage and self-compaction is in the vertical direction only. The very slight pressure needed for the final brooming operation is insufficient to upset the benefits derived from the mechanical compaction.

More efficient use of reinforcing steel is the aim of these recommendations for better identification and segregation of this material.

Economy Through Better Control of Reinforcing Steel*

By F. TESSITOR† and P. ROSEWARNE‡

SYNOPSIS

Difficulties and troubles encountered through the use of reinforcement steels not in strict accordance with present-day specifications are discussed. Data are presented to show the possibility of alleviating the situation for engineers, producers and contractors. Simplification of materials requirements, liberalization of code requirements to permit hot bending of bars and permanent identification of grade of steel would, in large measure, permit designers to apply reinforcement to structures in a more effective manner.

INTRODUCTION

Much time and effort are expended to obtain uniform concrete of the desired quality, but too little thought is given to the reinforcing steel used in many concrete structures. Present specifications permit the properties of steel reinforcing bars to vary over a wide range; moreover, the recent period of steel scarcity seemed to create a general letdown of quality.

Both Federal and ASTM specifications classify steels for reinforcing bars into three *types*: Billet steel in structural, intermediate and hard *grades*; rerolled axle steel in structural, intermediate and hard *grades*; and rerolled rail steel in hard *grade* only. The first two types of steel fall into their various grades as determined by their tensile properties.

These specifications have been in existence for a number of years, and seemingly few objections have been raised against reinforcing bars being separated into three grades. Yet, when a given lot is sampled, frequently all three grades of material are found. For example, tests were conducted on samples of 18 bars (from $\frac{5}{8}$ to $1\frac{1}{4}$ in.) which broke in bending in construction work on the All-American Canal in 1946. These tests revealed a range of carbon from 0.35 to 0.79 percent, with tensile strengths from 72,500 to 136,100 psi. Yet, the material was purchased as billet steel, Grade 2 (intermediate), in

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accordance with Federal Specification QQ-B-71a. The case is further illustrated by test results of approximately 20 specimens of ½-in. deformed bars received by the Denver laboratories of the Bureau of Reclamation. These bars were obtained in 1941 under the same specifications as the case cited above and were found by physical testing to be entirely of hard grade.

The foregoing cases illustrate one of the main deviations from specifications requirements. Evidently, steel mills are either failing to segregate their steels properly, their rolling practices create too wide a spread in physical properties, the steel warehouses cannot or do not maintain proper separation of grades and types, or field construction forces find it extremely difficult to segregate the bars into their respective grades.

Reinforcement is just as important as the ingredients that make up the concrete, or the steel I-beams, plates, or columns; yet it has not been handled or treated on a basis equal to that of other materials. To conserve on reinforcing steel required for any particular job, it is necessary for the designer to liberalize the working stresses used. Before he can do this, however, it is necessary to provide a dependable material, a provision which is not being fulfilled at the present time. Two existing specifications, ASTM and Federal, covering reinforcement, have some shortcomings; but, even perfect specifications do not accomplish their purpose if they are disregarded.

SOURCES OF MATERIAL FOR REINFORCING STEEL

The basic open-hearth process accounts for most of the plain carbon steel produced at the present time, and almost all reinforcing steel is made by this process. The specifications for new billet material make no mention of discarding the top of the ingots to eliminate pipe and segregation. The steel must meet a phosphorous maximum of 0.10 percent and a tensile and bend test. As there is no specified carbon range for billet steel, it can vary from very low in the structural grade to a higher carbon content than rail steel in the hard grade. Rail and axle steel are made predominantly by the basic open-hearth process and require sufficient top discard of the ingot to eliminate segregation and pipe. ASTM specifications provide a phosphorous maximum of 0.04 percent for rail steel and 0.05 percent for axles. Rail steel generally ranges from 0.60 to 0.80 percent carbon and is of hard grade. Most axle steel contains 0.40 to 0.55 percent carbon and if rerolled into reinforcing bars will normally produce material falling in the intermediate grade. Because steel producers may be held responsible for damage caused by defective rails or axles, a careful quality control is exercised in their manufacture, and the product is superior to steel made for structural or reinforcing purposes. The fact that the material is used as rails or axles before being rerolled into reinforcement should have no deleterious effect on the final product. The rerolling accomplishes a reduction of cross-sectional area, resulting in a refinement of grain structure and a more homogeneous material.

Bessemer steel is listed in the 1948 *Metals Handbook* as comprising about 5.6 percent of the total steel-making capacity. Although both the ASTM

and Federal specifications allow the use of acid Bessemer steel for reinforcement, its low carbon content should keep it in the structural classification, and practically none would be encountered by a consumer who specified intermediate grade.

SPECIFICATIONS FOR REINFORCING STEEL

ASTM specifications

ASTM specifications for reinforcing steel are so liberal in the number of tests required and retests allowed that practically any steel now produced would be acceptable. ASTM Specifications A 15-39 for billet steel bars, A 16-35 for rail steel bars, and A 160-39 for axle steel bars all contain the following statement: "If any test specimen develops flaws, it may be discarded and another specimen substituted." If this statement is interpreted broadly enough by the inspector, numerous tests could fail and be discarded until one satisfactory sample is found. Only one sample is required for an entire heat of steel which may be well over 100 tons.

A possible reason for wide range in physical characteristics of some lots of rerolled axle steel reinforcing bars is apparently due to disregard, by some mills, of the requirement of making carbon determinations and separation of axles into lots by carbon range. This condition makes it possible for steel ranging from structural to hard grade to be found in the same lot and classified according to the grade of the one test piece required for each lot. ASTM Specifications A 160-39, for axle steel bars for concrete reinforcement, cover this point quite well as follows:

> "The manufacturer shall make a determination for carbon of each axle received by him for manufacture into reinforcement bars. Based on these carbon determinations, all steel axles shall be stocked for subsequent rolling in separated lots by carbon range. The ranges of carbon shall be determined by the manufacturer as those best suited to meet the physical requirements. When requested by the purchaser, the manufacturer shall report the carbon range for each lot of bars furnished."

Carbon determination by spark test should be adequate for this purpose; accordingly, the manufacturer is in a better position to control the rolling temperature properly for each grade.

Federal specification

Federal Specification QQ-B-71a is more exacting as to the number of tests required and retests allowed; therefore, in general, steel which will pass the Federal specification may be expected to be of more uniform quality. The Federal specification requires one test for each 25 tons of steel from any one heat and makes no mention of retests of billet steel. Retesting of rerolled material is permitted as follows:

> "If any of the test specimens selected from lots of bars produced from either car axles or T-rails fails to meet the requirements, and a retest is permitted, at least four additional specimens shall be selected by the inspector. If any of the four specimens fail, the lot represented thereby will be rejected, and no further retests will be allowed."

USE OF HARD-GRADE REINFORCING

The greatest difficulty encountered by the Bureau of Reclamation in the use of reinforcing steel is in the hard grade, where breakage occurs during cold bending. This breakage has no effect on the structures because broken bars are discarded; nevertheless, it is an annoyance and an expense to the contractors. This condition was accentuated when steel was difficult to obtain, which is a major factor in the relaxing of quality controls by the manufacturer. An example of this condition follows: Seven types of deformed bars, varying either in size or design, were obtained for testing from the stockpile of reinforcement steel at one of the Bureau projects. Some bars which were rerolled axle and rail steel were identified accordingly. The balance was of unknown origin. Subsequent tensile tests revealed there were structural, intermediate and hard grades of axle in the lot, and that four of the seven types of steel would not pass the particular bending requirement for its grade. The yield strength of this mixed material varied from 36,500 to 126,200 psi as listed in Table 1.

Distrust of hard-grade steel

Some individuals are suspicious of incipient cracks and fear brittleness under impact; therefore, they are distrustful of hard-grade material. In a committee report, Gilkey and Ernst[1] discuss the possibility of sound bars being ruptured within a member under impact loads on the latter. They point out that the steel is packed in a matrix of concrete through which all stress must be applied, and since concrete is relatively friable and more brittle than steel, it seems inevitable that the concrete would have to be thoroughly shattered before steel of any grade could be damaged from impact through concrete. With reference to incipient cracks, Pearce[2] investigated results of testing rail steel bars from nine different sources and makes the following conclusion:

"In the general discussion of the ideal reinforcing bar, it was stated that the bar should break completely if it breaks at all. This is the case with high carbon bars such as hard-grade billet or rail steel reinforcing bars, so there can be no suspicion that such bars with

TABLE 1—PROPERTIES OF REINFORCING BARS

Type of bar	Grade	Yield strength, psi	Ultimate strength, psi	Percent elongation in 8 in.
1⅛-in. sq rerolled axle, deformed	Structural	36,500	65,800	*
1⅛-in. sq rerolled axle, deformed	Hard	52,400	96,000	*
1⅛-in. sq rerolled axle, deformed	Intermediate	46,000	81,700	26
1⅛-in. sq unknown origin, deformed	Hard†	126,020	141,700	*
1-in. round rerolled rail, deformed	Hard†	72,700	125,800	*
1-in. round rerolled rail, deformed	Hard†	72,700	146,000	6
⅞-in. round rerolled rail, deformed	Hard	66,200	124,000	6
⅞-in. round rerolled rail, deformed	Hard	66,000	128,000	9
⅞-in. round billet, deformed	Hard	66,400	113,500	13
⅞-in. round billet, deformed	Hard	63,600	114,100	15
⅝-in. round billet, deformed	Intermediate	49,400	83,500	23
⅝-in. round billet, deformed	Intermediate	49,200	82,700	17.8
⅝-in. round unknown origin, deformed	Intermediate†	47,000	78,600	21.5

*Broke out of gage marks.
†Failed on bend test.

partial fractures will get into the structure. With the lower strength materials, it may be possible to have a fractional failure without total breakage."

Bending of bars

In some cases of breakage of hard bars, contractors have expressed a willingness to heat the bars for bending to expedite construction. Permission to hot bend has been refused by the field engineers in some cases, because most codes contain the stipulation that reinforcing bars be bent cold. Tests made in the Denver laboratories on ½-in. round deformed rerolled rail steel with a hardness of Rockwell B98 showed no difference in hardness between the bar as rolled, cold bent around a pin of 2-in. diameter, or hot bent around the same pin and air cooled. To illustrate this point further, a 1⅛-in. round rerolled rail steel bar was cut into three lengths. One piece was cold bent in the as-rolled condition 90° around a pin having a diameter 4 times that of the bar. One was normalized at 1550 F for 3 hours and then cold bent on 4 diameters. The third was bent at a red heat on a sharper radius and air cooled. The hot-bent bar was locally heated with a torch for about 3 in. of its length to accomplish the hot bending. The bent portion of each bar was then sectioned and hardness determinations made as illustrated in Fig. 1. The two cold-bent bars registered a slight increase in hardness on the inside of the bend, due to the compressive stresses set up in bending. These stresses were eliminated in the hot-bent bar and there was no noticeable difference in hardness. It is believed that hot bending should not be prohibited by the codes if the expense of heating is warranted and the contractor takes precautions not to cool the bars too rapidly, as by water quenching or by dropping the hot bars into snow. Wyss[2] makes the following recommendations:

"In the interest of safety it should be specified that for bars having a diameter of 1⅛ in. or more, all end hooks should be made by hot bending. Wherever bends sharper than 5 times the diameter of the bar are required, they should be made only by hot bending."

One lot of reinforcing steel purchased for construction at the Denver laboratory produced samples of rerolled rail steel which were too brittle to bend 90° as required by the specifications, but could be cold bent 180° without breaking after a 3-hour normalizing treatment at 1550 F. This is illustrated

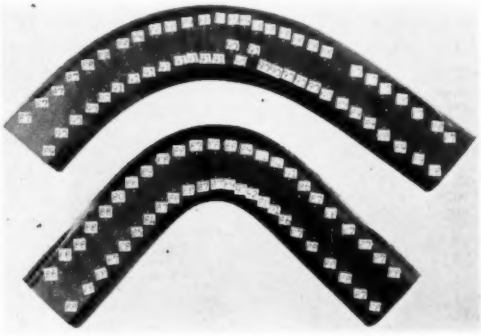

Fig. 1—Rockwell G hardness readings on cold bent bar above, and hot bent bar below. Hardness figures obtained on a cold bent normalized bar are practically identical to the bar above, cold bent from the as-rolled condition.

Fig. 2—Three samples from the same bar of 1⅛-in. rerolled rail steel. The broken bar shows typical results of cold bending the bar as rolled. The other two specimens were normalized by heating for three hours at 1550 F and cooled in still air. The center bar is bent 90°, as required by the specifications. The top bar was bent to the limit of the bending fixture in an unsuccessful attempt to break it.

in Fig. 2. It is believed that a more careful control of rolling temperatures and rate of cooling at the mill could accomplish the same results. Other bars of rerolled rail received by the laboratory evidenced marked overheating.

The writers made a number of cold bends of rerolled rail steel, some of which were too brittle to pass the bending requirement of 90°. The bends were made in the apparatus illustrated in Fig. 3 which allows bending without frictional restraint. These bent bars were sectioned as illustrated in Fig. 4 and carefully examined before and after a deep etch with hot *HCl*. No cracks could be found in any specimen which did not break and only the one at the point of failure in the specimens that did break. A typical break produced in hard material is illustrated in Fig. 5. Hence, the authors are convinced that if a crack develops in this material, the bar breaks completely.

ECONOMICAL USE OF STEEL

Other users of structural steel have found it economical and desirable to use higher tensile steels. Reinforced concrete designers may also follow this trend. The main drawback to this application at the present time is the lack of uniformity and means of positive identification of the various grades. Changes in practice such as closer control of rolling temperatures, rate of cooling and means of keeping the various grades separate would help con-

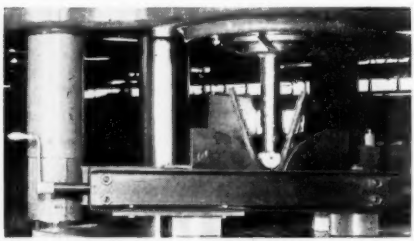

Fig. 3—Scholer-type fixture used in making bend tests. The quartered rollers are free to move so they will not introduce restraint in the bar.

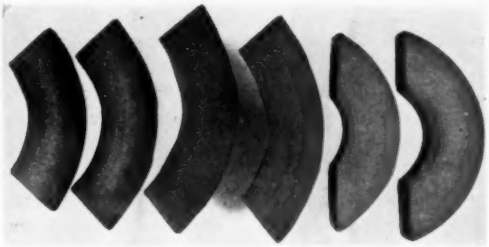

Fig. 4—Specimens of 1-in. sq re-rolled axle steel, showing the manner of preparing the bent bars prior to inspection for cracks. They were deep etched in hot 50 percent HCl.

siderably to conserve steel. The construction personnel should definitely ascertain that the material being used will meet the designer's requirements. Proper quality control and inspection will have to be demonstrated to the designer before he can be expected to reduce his safety "ignorance" factors by using higher working stresses to conserve steel.

If working stresses of hard bars are to be increased, better identification of bars is necessary. Painting the ends is inadequate if the bars are cut up or stockpiled long enough to rust. It would be advantageous to the consumer to have the mills roll an identifying mark such as *S*, *I*, and *H* into each bar to signify that it is of structural, intermediate or hard grade. This operation should not be an undue hardship on mills rolling new billets since the analysis of each heat is known. Furthermore, it should not be a hardship on rerolling mills complying with ASTM specifications on axle steel, which require segregation of lots by carbon content. (Of course, all rerolled rail is of only one grade.)

The present classification of seven types and grades of steel could be easily reduced to three: namely, structural; intermediate; and hard. There seems to be little need to differentiate between new billet and rerolled material if the rerolled steel can be properly classified and identified as to grade.

Predicated on proper inspection and identification of steel by grade, there appears to be considerable room for economizing in the use of reinforcement. It seems inconsistent that a working stress of 18,000 psi is used for steel with

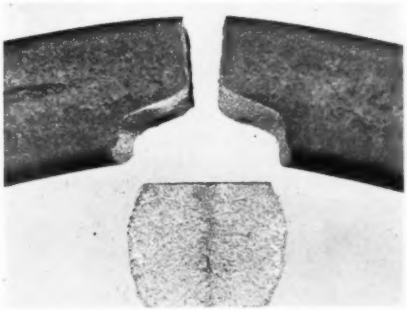

Fig. 5—Typical "pop out" break produced by high compressive stresses on inner radius of bend during bend test of hard steel

a minimum yield point of 33,000 psi, while a working stress of only 20,000 psi is used for steel with a minimum yield point of 50,000 psi. The average yield point of the hard grades is more likely to be over 60,000 psi. It is apparent that a large proportion of this steel is not being put to work effectively. It has been suggested by a number of writers that the working stress of hard-grade bars be increased to 24,000 psi as was done in emergency specifications during the last war.

Under certain market conditions, rerolled rail and axle steel is cheaper than billet steel; and the use of this material would allow an additional saving if higher unit stresses were used. This saving would also apply to hard grade billet steel which is no more expensive than structural grade.

Any reduction possible in the tonnage of steel used subsequently reduces the cost of fabricating and placing of the steel and facilitates placing of the concrete. Some cases have been noted where the steel is so closely spaced that it is difficult to place the aggregate.

These recommendations are based entirely on the physical properties of the steel, and do not include consideration of bond stress or area of reinforcement. The highly deformed bars now being produced according to ASTM Specifications A 305-49 in increasing quantities may permit the use of less steel from a bond strength standpoint.

SUMMARY

As a result of this investigation, it is believed that the reinforcing steel situation could be improved greatly by the following steps:

1. Permit hot bending of any bars
2. Consider only three grades: structural, intermediate and hard
3. Encourage manufacturers to permanently identify the grade of steel in each lot
4. Conduct more rigid inspections
5. Specify a reasonable number of tests per heat or lot and limit the number of retests

If the above points are accomplished, the designers can effectively use higher design stresses which would result in considerable savings in material and handling costs.

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Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

The problem of prestressing redundant structures (Il problema della pre-compressione su solidi vincolati iperstaticamente)

GIUSEPPE RAINERI, *Giornale del Genio Civile* (Rome), V. 87, No. 11, Nov. 1949, pp. 595-602
Reviewed by GENNARO MIANULLI

The author investigates a few limited cases of continuous beams susceptible to prestressing. He establishes equations for the general case only, and in relation to the line of the cable. The possibility of obtaining the center of prestressing outside the section with internal cables is discussed.

Concrete fences

A. M. PENNINGTON, Concrete Publications Ltd., London, 1950, 66 pp. 6 shillings

British design, manufacture and erection of concrete fencing materials ranging from posts for wire fences to all-concrete post and panel fences are described in detail and reflect the extensive use of concrete for this purpose in Britain. Although most of the fencing methods described are not common practice in this country the book nevertheless contains much helpful information for those interested in using concrete to replace scarce and short-lived wood for fences.

The new Saint Nicolo bridge—Florence (Il nuovo ponte di S. Nicolo in Firence)

RICCARDO MORANDI, *Giornale del Genio Civile* (Rome), V. 87, No. 12, Dec. 1949, pp. 628-630
Reviewed by GENNARO MIANULLI

Hollow sections were used in a reinforced concrete bridge erected on the river Arno in the Saint Nicolo section in Florence to replace a 3-span girder bridge destroyed in June 1944. It has 8 hollow arch ribs 91 m long, and 8 cm rise, and ribs spaced 2.90 m center to center; crown depth of 132 cm increasing to 350 cm at the skewback, with web and flange thicknesses varying from 30 to 40 cm, tied by diaphragms. The roadway supported on spandrels is designed and built to relieve the stiffening of the arches.

The calculations agreed with the results obtained from test on the completed structure under full live loading.

A method of determining quick hardening of mortar and pozzolana pastes (Sistema per accelerare l'indurimento degli impasti di calce e pozzolana)

FERNANDO PARISSI, *Giornale del Genio Civile* (Rome), V. 87, No. 10, Oct. 1949, pp. 534-538
Reviewed by GENNARO MIANULLI

A system tested by the author can be used advantageously in the construction of harbor and river substructures. Data are presented in the form of tables and graphs. As these tests are not conclusive, a more extensive program is proposed and additional tests are to be made in the laboratory of the engineering school at the University of Rome.

Effect of skew angle on rigid-frame reactions

WALTER C. BOWER, *Proceedings*, ASCE, V. 76, Separate No. 32, 1950, 12 pp. $0.50

Because of doubt of the usefulness of skewed bridges brought about by past failures in two-hinged, rigid-frame bridges an investigation was made of models to confirm the validity of the analytical method of

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solution for reactive forces. The variation of these forces was studied for a complete range of skewed angles with an Eney deformeter. The centerline influence ordinates showed the greatest variation between model and analytical results. Since these ordinates are normally used in design, theory will tend to give results substantially below model results. This deviation becomes pronounced as the skew angle becomes large.

The calculated windforce in structural design (Der Berechnungswind in der Baustatik)

W. A. WOEBER, *Zeitschrift des Oesterreichischen Ing. und Arch. Vereines*, V. 95, No. 7-8, 1950, pp. 53-59
Reviewed by RUDOLPH FISCHL

The wind stress design required by most codes is based on Newton's theory of resistance but does not take into account the shape and roughness of the resisting surface or of the turbulence of the wind flow.

Wide fields are still open for research. The author gives new aspects by discussing the aerodynamic fundamentals of tests made in wind tunnels, meteorological conditions, registrations, intensity and period of wind, variations of wind forces with increasing height over the ground and other interesting subjects. This paper points out how necessary it is to provide the practicing engineer with better data for wind design.

Materials of construction

JOHN H. BATEMAN, Pitman Publishing Corp., New York, 1950, 568 pp. $6.50

Designed as an undergraduate text book, inclusion of information on all the common construction materials necessarily leads to brevity and generalized treatment. Despite this, almost half the book is devoted to concrete and its constituents. Of interest to those specializing in concrete are chapters on mineral aggregates, cementing materials, concrete materials, properties and proportions, and the manufacture and placing of concrete.

The origin, production, shipment, handling and storage of aggregates are briefly described while the physical properties are covered in somewhat more detail. The factors affecting the properties of concrete are considered at some length and proportioning is outlined. Methods of mixing, placing and curing concrete are given and special procedures such as vacuum concrete and prepacked concrete are described.

The book is written in a clear, easily understood manner and should be of interest to the beginning engineer as well as the student. An extensive bibliography lists sources of further information for those who wish to learn more about materials.

The relation of thermal expansion of aggregates to the durability of concrete

EDWIN J. CALLAN, *Bulletin* No. 34, Waterways Experiment Station, Vicksburg, Miss., 28 pp. $0.50

Procedures developed to investigate the relation of thermal expansion of coarse aggregates and mortars to the durability of concrete as measured by resistance to accelerated freezing and thawing are described. By the use of these methods information was obtained on the linear coefficient of thermal expansion of component materials in the concrete. Statistical analysis of these data demonstrated that for certain concretes there is a significant relationship between durability and thermal expansion of the components.

In addition to the description of the test methods and suggestions for use of the data to explain the lack of durability of materials otherwise thought to be sound, caution is indicated in the use of thermal data for predicting the durability of concrete.

Symposium on use of pozzolanic materials in mortars and concretes

Special Technical Publication No. 99, American Society for Testing Materials, Philadelphia, 208 pp. $2.50

This syposium presents a comprehensive picture of the present status of this subject. There has been a tremendous upsurge of interest in these materials in the past decade. This publication should be of service to all concerned with the production and use of cement and concrete and the design of concrete structures.

Following an introduction by T. E. Stanton, the compilation contains the following nine papers: Pozzolanic Materials and Their Use in Concrete—R. E. Davis; Pozzolans Used in Mass Concrete—H. S. Meissner; Expérience with Pozzolanic Materials in Kansas—C. H. Scholer and R. L. Peyton; Effect of Calcination on Natural Pozzolans—R. C. Mielenz, L. P. Witte and O. J. Glantz; Effect of the Use of Diatomite

Treated with Air-Entraining Agents upon the Properties of Concrete—R. E. Davis and A. Klein; Summary of Methods for Determining Pozzolanic Activity—W. T. Moran and J. L. Gilliland; Strength, Volume Change and Sulfate Resistance of Mortars Containing Portland-Pozzolan Cements—R. E. Davis, W. C. Hanna and E. H. Brown; Studies of Some Method of Avoiding Expansion and Pattern Cracking Associated with the Alkali-Aggregate Reaction—W. C. Lerch; Studies of Use of Pozzolans for Counteracting Excessive Concrete Expansion Resulting from Reaction Between Aggregates and Alkalies in Cement—T. E. Stanton.

Progress in reinforced concrete (Fortschritte im Stahlbetonbau)

RUDOLF SALIGER, Franz Deuticke, Vienna, 1950, 140 pp. $3.20

Reviewed by WERNER H. GUMPERTZ

The properties and performances of deformed reinforcing bars are discussed in detail by this well-known Austrian engineer. Deformed bars appeared later in Europe than in this country, and have been subjected to extensive tests during the last decade.

Some significant results of European studies and experiments are listed, particularly the following:

1. The strength of reinforced concrete depends on the yield point of the reinforcing steel—in case of tension failure, directly; in case of compression failure, through shifting of the neutral axis due to yielding of the steel.

2. Cold worked steel bars have a yield point which is higher relative to the ultimate strength and more indefinite than ordinary (plain) bars, allowing a better utilization of steel strength in reinforced concrete.

3. Prestressed concrete requires higher strength bars because of the shrinkage which is to be expected.

4. Overstressing of steel bars should cause cracks no greater than 0.004 in. for exposed structures, and 0.008 in. for others.

5. Many small cracks are better than few large ones. Higher concrete strength reduces size of cracks. Cracks due to a stress of 43,000-57,000 psi in some types of deformed bars are not larger than cracks due to a stress of 17,000-20,000 psi in smooth bars which are now predominantly used in Europe.

6. There is nothing to be gained by increasing the bonding strength of steel bars beyond the shearing strength of the concrete cylinder enveloping the extreme portions of the deformed bar.

Prestressed concrete bridges over the Marne river (5 bruggen in voorgespannen beton over de Marne)

A. STARINK, *Cement* (Amsterdam), No. 17-18, 1950, pp. 376-382

Reviewed by J. W. T. VAN ERP

These 5 bridges were built to replace those destroyed during the war. They all have the same span of 235 ft and are of identical construction using prefabricated parts exclusively, except in the abutments. Each bridge consists of 6 slightly arched I-section girders of extreme slenderness. These girders are composed of prefabricated sections, 7 to 8 ft long made in a nearby fabricating yard. They were cast in double-walled vibrated steel molds and the concrete was immediately compacted by a pressure on the top surface of 70 psi. After a 2¼-hour heating period the concrete, having attained a strength of 2200 psi allowable compressive stress, was removed from the forms. These sections were then assembled in groups of 5 or 10 into temporarily prestressed elements. This provisional prestressing served only to facilitate transportation to and erection at the site. Groups of cables were prestressed at the outside of the webs of the I-sections, to be removed after the final steel was in place and prestressed. The beams were swung into place from steel towers at each abutment; no scaffolding whatsoever was used. The girders have a height in the center of only 34 in., making for this extremely slender structure a depth/span ratio of only 1/84. That the structure is at the same time economical in materials is shown by the fact that for 6400 sq ft of bridge deck 460 cu yd of concrete, 10.7 tons of high tensile steel and 6 tons of mild steel were used. The bridges were designed for a load of 125 psf on the roadbed and 82 psf on the sidewalks.

The main difficulty that still holds back a widespread application of prestressing in this country is the relatively great amount of skilled labor needed. However, these structures demonstrate some of the typical ad-

vantages of prestressed concrete construction: (1) extreme lightness of construction comparing favorably with steel construction, (2) concrete quality as excellent as can technically be made, (3) economy on formwork, (4) total absence of shrinkage or cracks in the concrete and (5) no scaffolding.

Properties of lightweight concrete (Korrelbeton)

Otto Graf, *Cement* (Amsterdam), No. 21-22, 1950, pp. 457-459

Reviewed by J. W. T. Van Erp

The ever increasing use of lightweight concrete for wall construction in Western Germany is due largely to the huge quantities of rubble available since the war. This rubble is a suitable aggregate consisting chiefly of broken stone and only a minor proportion of brick. Besides alleviating the scarcity of building materials, it also leads to more efficient building construction than would have been possible with traditional masonry wall construction. Thus, in more than one way, the use of rubble as a lightweight aggregate fitted well into the reconstruction program. In line with this, an extensive research program was carried out at the Stuttgart Institute of Technology covering:

1. Properties of the aggregate, its granular structure and proportion of voids. Weight of the concrete at 28 days, dry, ranged between 78 and 94 lb per cu ft with corresponding test-cube strengths of 280 psi and 1130 psi respectively. Weight of concrete proved to be about 1.1 times the sum of the weights of aggregate and cement.

2. Properties of cement and cement content. High early strength cement proved to be most satisfactory so that forms could be removed under normal weather conditions after one or two days. A cement content of 6.2 to 7.5 lb per cu ft yielded concrete with a 28-day strength of 430 psi.

3. Consistency of cement mix, to provide a workable degree of plasticity, required the cement mortar to contain so much water that it just failed to flow out of the mix. This actually resulted in concrete of greater strength than with stiffer cement mortars. It provided for a better bond between the aggregate particles than could be obtained by a direct contact between the aggregate parts. The surface of the concrete, after hardening, thus presented the aggregate particles coated with a hard glazelike coating of cement-mortar.

4. Methods of batching of the aggregates and concrete mixing.

5. Depositing concrete, which proved to be done best in thin horizontal layers with only light prodding.

6. Curing of concrete.

7. Relation between weight and strength of concrete.

8. Actual strength of walls, which proved to be always less than that of test cubes. The actual strength of 8-ft high walls proved to be only about half of test cube strengths.

9. Strength of walls under concentrated loads.

10. Continued testing of aging concrete.

11. Shrinkage.

12. Thermal conductivity and sound-proofing quality. For partition walls in dwellings 8-in. thickness proved satisfactory for usual requirements.

Modern structures in Sweden (Ouvrages d'art modernes en Suede—Rapport)

G. De Cuyper, *Annales des Travaux Publics de Belgique*, Feb. 1950, pp. 53-73

Reviewed by Charles C. Zollman

Sando bridge

What in the opinion of this reviewer are some of the most impressive and aesthetically beautiful reinforced concrete structures erected in recent years, are described in detail and well illustrated in this report.

To take advantage of excellent river bank rock foundations and to provide a 130 ft navigation clearance above mean water level, Swedish engineers decided upon an open-spandrel hingeless arch of almost 900 ft span and 130 ft rise. The arch is flanked at one end by a 670 ft trestle and at the other end by a 1150 ft trestle; resulting in a total length of bridge of about 2700 ft. Basically, the rib is a box girder 33 ft wide, with depth varying from 10 ft at the crown to 16 ft 6 in. at the springing line. The top and bottom slab of the box-girder, and the 4 vertical longitudinal walls (2 exterior and 2 interior) connecting the slabs are all only 12 in. thick.

The 20-ft roadway is flanked at either side by a 6 ft wide bicycle path and a 4 ft wide cantilevered sidewalk. The 7 in. thick carrying slab transfers the loads to three longitudi-

nal rectangular reinforced concrete girders which are connected every 40 ft by transverse concrete stiffeners. These stiffeners transfer the loads to two circular columns which are also 40 ft on centers and which bear on the interior longitudinal walls of the rib. These interior walls are 20 ft on centers, symmetrical about the centerline of the arch. While the author elaborates on these data, the most interesting feature of the report concerns the collapse of the first timber centering which occurred right after the concrete at the crown was poured for the bottom slab of the rib.

After analyzing the possible causes of this collapse, the second and final centering, used with success, is described. A brief description of the approach trestles concludes this first section of the report.

Kockestrand bridge

This bridge is less spectacular than the Sando arch bridge but presents some unusual design features. The main section of the bridge is a variable depth continuous three-span girder plus an overhang. The interior span is 235 ft, each of the two exterior spans 130 ft, and each cantilever extends beyond the supports 18 ft.

The cantilevers receive the first of a series of 60 ft simple supported approach spans. The total length of the bridge including the approach spans, is about 1000 ft. Only the continuous girder is described. To reduce the negative moments over the supports, and thus at the same time reduce the dead weight and reinforcing steel required, the Swedish engineers simply decided to induce some compression in the structure by prestressing. Eighty-four 1¼-in. diameter steel bars having an elastic limit of about 52,000 psi and stressed to 26,000 psi, induced a compressive force of about 1200 U. S. tons. Not only were the moments reduced by 20 percent, but at the same time principal tensile stresses due to shear were appreciably reduced. The negative moments could have been entirely eliminated had it been possible to induce a compressive force of 3500 U. S. tons. Unfortunately, high tensile strength wires capable of being stressed to 80,000 psi were not available at the time. Another interesting feature is that the four supports of the continuous girders are fixed end supports. Only the cantilevers are provided with expansion joints; the longitudinal deformations are thus taken up by the pile bents.

The author notes that the concrete surfaces required no surface treatment because formwork was prepared and installed with extreme care. No construction joints were needed as the concreting schedule permitted continuous placement. Where possible and economically feasible, steel forms were used. Energetic vibrating of the concrete brought laitance to the surface and this gave the concrete the aspect of white marble. Color was uniform as the same cement from the same factory was used throughout the entire job.

Economically the structures were within reasonable range, particularly because timber was to be found right on the spot and rock foundations were practically on the surface.

The Construction of Clark Dam on the Derwent River, Butler's Gorge, Tasmania

J. A. Slayter, W. L. Godfrey, and M. C. Heffernan, *Journal of the Institute of Engineers*, Australia, V. 22, No. 1-2, Jan.-Feb. 1950, pp. 1-24

Reviewed by J. R. Shank

Clark Dam is an arch-gravity structure on a circular arc of about 120 degrees. The dimensions at the crown are 225 ft from water crest to tailwater, 83 ft thick at the base and 15 ft thick at the top.

Dolorite rock quarried within a half-mile of the site was used for aggregate. This is a kind of basalt and is very hard and tough. Crushing was exceptionally severe on crushing equipment and much had to be learned about maintenance. The aggregates were (1) cobbles, 1¾ to 3¼ in., (2) coarse rock, 1 to 1¾ in., (3) medium rock, ¾ to 1 in., and (4) sand. The fines under ¾ in. from the crushing for the coarse aggregate were not used because of their flat, flaky, angular shapes. The sand had to be reduced from larger rock by an entirely different crushing process.

The cement came in paper bags. A centrifugal bag-holding device with double-knife bag slitter and a special turning-over mechanism reduced cement bag wastage to less than one-half percent. The cement was a mixture of low-heat and normal cement with no air-entraining agent or other admixture. The cement had a surface modulus of 1850 sq cm per g by the Wagner turbidimeter. A minimum cement factor of 444 lb of cement per cu yd and a water-cement ratio of 0.65

was fixed by specifications. Cores from 90-day old concrete gave compressive strengths of 6000 psi, though laboratory cylinders of the same concrete gave only 4000 psi.

Cantilever timber forms were supported by bolts in the last two lifts of 5 ft each. Fluid pressures were figured for 5-ft heads. Waterproof plywood was used for exposed surfaces. Wooden forms with plywood linings were used as many as 35 times. Concrete was transported by narrow-gage track to a spot under the cableways. Two 10-ton electrically operated cableways covered the main site. The towers were 1200 ft apart and the whole cableway system could travel 400 ft laterally. Operational instructions were given entirely by radio. Radio transmitters were so situated as to enable the operator to hear the mechanism working and detect improper operation. The main cables were of a semi-locked coil construction having a circumference of 7¾ in.

Concrete was placed with the aid of Ingersoll-Rand, two-man, 80-lb, 3 V internal vibrators. The minimum stripping time was 60 hours. When the initial set had taken place, 3 to 4 hours after placing, the surface was hosed with water from a fire-hose at high velocity. The laitance and surface fines were washed into a sump formed at the center of the block. Before new concrete was placed, mortar at least ½ in. thick was spread on the surface of the old and broomed into the surface. Horizontal finished surfaces were covered with a layer of sand which was kept moist for approximately 3 weeks after pouring.

The mean annual weather temperature was 44 F. The placing temperature varied from 45 to 65 F. The initial heat rise in the first few days was 25 to 30 F. The concrete had to be cooled from a maximum of 95 F in summer and 75 F in the winter. Cooling coils carrying river water were cast into each lift.

A careful program of pouring was worked out from one to three months ahead so as to control the crack openings between blocks and to give time for changing forms. A gap of 0.03 in. was considered to be the minimum for grouting. The operation of cooling water and occasionally heating water was carefully worked out to get the desired crack openings.

The preliminary grouting, both blanket and curtain grouting, was at relatively low pressures, 25 to 35 psi. Blanket grouting was done through a 10 ft cover of concrete.

The cement for grouting the joints was cleaned of coarse particles of clinker and coarse foreign material by means of an air separator. The first grout batches in all cells were 2 to 1 water-cement ratio by volume, but it was changed to 1 to 1 or 0.9 to 1 when the grout reached the top of the cell. It was found that the blocks were remarkably flexible and that the gap gage at the crest could be used most accurately to control the size of the joint being grouted.

The Michigan Test Road—Nine year progress report

Michigan State Highway Department, Report No. 137, Aug. 1950, 74 pp.

A nine year progress report of the Michigan State Highway Department on the Michigan Test Road experimental project reveals several conclusions of outstanding importance in the design and construction of modern concrete highways.

The test road was constructed in 1940 in cooperation with the Bureau of Public Roads and is one of six such test roads, the others being in California, Kentucky, Missouri, Minnesota and Oregon. It is divided into two experimental sections. One section, designated the Design Project, is 10.1 miles long and coincides in a general way with the Bureau of Public Roads plans and procedure for the construction of experimental roads. The other experimental section, called the Durability Project, is 7.7 miles long and was included as a supplement to laboratory studies on the durability of concrete, especially in regard to scaling.

Studies made on this experimental road covered most of the basic problems of research investigation which affect concrete pavement design, such as the spacing of expansion and contraction joints, the maximum amount of steel reinforcement necessary and the relation of cross section design to the supporting value of the subgrades. Other factors included in the study were those which affect the durability of concrete and which involve the mixture formula and mixing methods as well as the chemical characteristics of the concrete used.

The most outstanding contributions resulting from the Test Road studies to date are the use of air-entraining concrete for scale prevention, the use of bituminous-rubber joint seal materials, the change to long slabs with heavier steel, and the elimination of expansion joints, except at designated locations.

Among the most important conclusions resulting from the experimental project are:

1. The satisfactory performance of long sections of pavement under full restraint indicates that expansion joints are unnecessary except at such places as intersections, railroad crossings and structures, where excessive compressive stresses introduced by expansion forces are undesirable.

2. Adequate load transfer devices are essential in all joints to preserve the mutual elevation of the abutting slabs.

3. The method of forming a contraction joint by grooving the surface of the pavement and subsequently filling the groove with a good sealing compound is far superior to inserting a premolded bituminous fiber strip.

4. Continuous plate dowels are not satisfactory as load transfer devices.

5. Commercially available asphalt-rubber joint-sealing compounds have greater durability than the mixed-on-the-job asphalt-latex mixtures and are far superior to the straight asphalt or tar products in common use for sealing joints and cracks.

6. Mechanical tamping of soil under forms produces more positive and uniform results than hand tamping.

The results of the studies show no indication that short slab construction is superior to long slab construction, but that many advantages are to be gained by the latter practice through better riding qualities, lower maintenance costs and better construction methods. Results do indicate, however, that the uniform cross section is equal in performance to that of the thickened edge section, with many obvious advantages.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirability of publishing a list of those periodicals being regularly scanned for articles dealing with the field of concrete. Included also are the publishers' addresses for convenience should correspondence with them be desired.

Annales de L'Institut Technique du Batiment et des Travaux	28 Boulevard Raspail, Paris 7, France
Bautecknik, Die	Hohenzollerndam 169, Berlin-Wilmersdorf, Germany
Betong	Brunkebergstorg 15, Stockholm, Sweden
Beton og Jernbeton	Laboratoriet for Byningsteknik, Oster Volgade 10C, Copenhagen K, Denmark
Beton-und Stahlbetonbau	Hohenzollerndamm, 169, Berlin-Wilmersdorf, Germany
Beton-Teknik	Kalvebod Brygge 2, Copenhagen V, Denmark
Betonstein Zeitung	Bingerstrasse 14, Wiesbaden, Frankfurt Am Main, Germany
Bulletin, ASTM	1916 Race St., Philadelphia 3, Pa.
Bulletin, Institution of Civil Engineers in Ireland	35 Dawson St., Dublin, Ireland
Bulletin, University of Illinois	Urbana, Ill.
Cahiers du Centre Scientifique et Technique du Batiment	33 Rue Jean-Goujon, Paris 8, France

Cement	Sarphatistraat 54, Amsterdam C, Netherlands
Cement and Lime Manufacture	14 Dartmouth St., London (Westminster S.W.1), England
Cement och Betong	Malmo, Sweden
Cemento, Il	9 Via Settembrini, Milan, Italy
Cemento Portland	Calle San Martin 1137, Buenos Aires, Argentina
Civil Engineer Corps Bulletin	Bureau of Yards and Docks, U.S.N., Washington 25, D. C.
Civil Engineering	33 W. 39th St., New York 18, N. Y.
Civil Engineering and Public Works Review	154 Fleet St., London E.C. 4, England
Commonwealth Engineer, The	349 Collins St., Melbourne, C. 1, Australia
Concrete	400 W. Madison St., Chicago 6, Ill.
Concrete and Constructional Engineering	14 Dartmouth St., Westminster S.W.1, England
Constructional Review	14 Spring St., Sidney, N.S.W., Australia
Crushed Stone Journal, The	1415 Elliot Place, N.W., Washington 7, D. C.
Current Road Problems	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.
Giornale del Genio Civile	Via delle Vite, 13, Rome, Italy
Engineering Experiment Station News	Ohio State University, Columbus, Ohio
Engineering News-Record	330 W. 42nd St., New York 18, N. Y.
Hormigon Elastico	Lavalle 1171, 5°, Buenos Aires, Argentina
Handlingar (Proceedings)	Ingeniors Vetenskaps, Akademien, Stockholm, Sweden
Ingenioren	Copenhagen, Denmark
Journal of the American Ceramic Society	2525 N. High St., Columbus 2, Ohio
Journal of Research	National Bureau of Standards, Washington 25, D. C.
Magazine of Concrete Research	52 Grosvenor Gardens, London S.W.1, England
Osterreichische Bauzeitschrift	Mölkerbastei 5, Vienna 1, Austria
Polytechnisch Tydschrift	Haarlem, Netherlands
Proceedings, ASCE	33 W. 39th St., New York 18, N. Y.
Public Roads	Bureau of Public Roads, U. S. Dept. of Commerce, Washington 25, D. C.
Reinforced Concrete Review, The	94-98 Petty France, London S.W.1, England
Revista de Obras Sanitarias de la Nation	Charcas 1840, Buenos Aires, Argentina
Revue des Materiaux	19 Rue Lafayette, Paris 9, France
Rock Products	309 W. Jackson Blvd., Chicago 6, Ill.
Structural Engineer, The	2 Bream Bldgs., London E.C.4, England
Technique Moderne-Construction, La	92 Rue Bonaparte, Paris 6, France
Teknisk Tidskrift	Kungsgatan 37, Stockholm C, Sweden
Teknisk-Ukeblad	Oslo, Norway
Travaux	6 Ave. Pierre 1st de Serbie, Paris 16, France
Zement-Kalk-Gips	Biebricher Strasse, 74, Wiesbaden, Germany